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THE INSTITUTION OF
CIVIL ENGINEERS

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AUGUST 1952

WORKS CONSTRUCTION DIVISION MEETING

12 February, 1952

Mr D. M. Watson, B.Sc., Vice-President I.C.E., Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Works Construction Paper No. 20

"The Construction of Loch Sloy Dam"

by

James Stevenson, B.Sc., A.M.I.C.E.

SYNOPSIS

The Paper is descriptive of the Contractor's methods, and of the problems encountered in construction of Sloy Dam.

The temporary works included such items as twin 10-ton aerial cableways, a 4-cubic-yard batching plant, and an aggregate conveyor.

An earth cofferdam was used to protect the low-lying excavations, and the flow from the loch was diverted along a concrete flume built through the cofferdam and the dam wall.

Excavation difficulties included dealing with a major fault which was not anticipated.

A brief description of pressure-grouting methods is given.

Welding of some reinforcement was carried out by the " Secrom " process.

Much of the more complicated work at the higher levels was greatly simplified by pre-casting units on the ground, and later hoisting these into position by means of the cableways.

Steel shutters were used throughout and a brief description of these and of the constructional details of the buttresses is given.

Some general notes are added on concreting progress.

INTRODUCTION

The mountains to the north-west of Loch Lomond are noted for their rugged splendour but perhaps more particularly, to the hydro-electric engineer, for their very high rainfalls which annually reach figures well in excess of 100 inches.

The Loch Sloy Power Project (described elsewhere [1, 2]) has been constructed here on behalf of the North of Scotland Hydro-Electric Board, to utilize much of this rainfall at a high head and produce a peak-load station of 130 megawatts installed capacity. The project is conveniently close to the large power-consuming areas of Scotland to which, it is estimated, it will supply an average of 120 million units annually.

To obtain the required regulation of the necessarily large flows, which reach 2,200 cusecs when on full load, a reservoir of 1,200 million cubic feet storage capacity has been created by the construction of a massive concrete dam at the outlet end of Loch Sloy which, when fully impounded, raises the loch level by 150 feet.

The elevation of the original Loch Sloy was 783·00 O.D., it being impounded by a localized upthrow of rock across the otherwise steeply sided and shaped valley, the hills on either side rising a further 2,000 feet. It is on this rock upthrow that the dam is sited.

For almost the entire period of construction, work on the site was greatly hampered by appalling weather.

During the years of construction, 1947, 1948, and 1949, rainfalls of 106 inches, 163 inches, and 153 inches respectively were recorded, giving a weekly average of 2·75 inches. The maximum weekly rainfall recorded was 11·36 inches and the maximum daily rainfall 4·10 inches.

The site was also subjected to a cold prevailing north-west wind of considerable force, and severe gales occurred with disheartening frequency.

THE PERMANENT WORKS

Loch Sloy Dam is of the mass-concrete buttress type, 1,200 feet long, has a maximum depth of about 160 feet, and contains 208,000 cubic yards of concrete. It consists of thirteen solid buttresses each 26 feet in width placed at 65-foot centres, with solid gravity-dam sections at either end running into the hillsides. (See *Fig. 1* and Fig. 2, Plate 1.) The vertical parallel sides of the buttresses fan-out in elliptical profile in plan at the upstream end to form the dam wall, the upstream face of which is inclined at 1 in 40 to the vertical as is the plane of the major axis of the elliptical face between buttresses. The downstream face of the buttresses slopes 8

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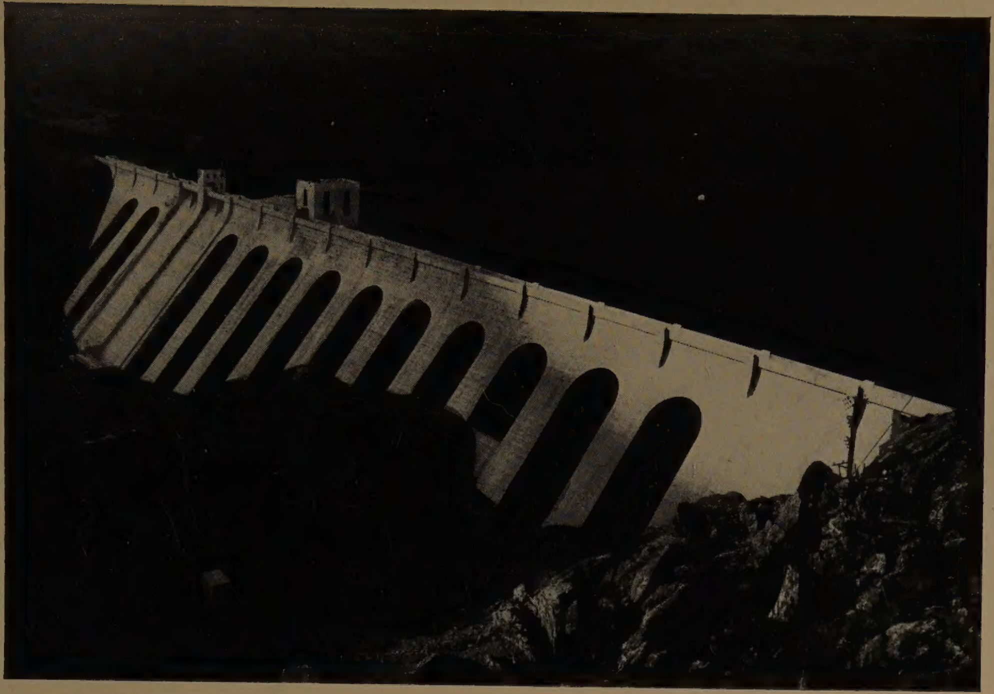
[1] "The Loch Sloy Scheme of the North of Scotland Hydro-Electric Board." *Engineering*, vol. 170, p. 1 (7th July, 1950), p. 25 (19th July, 1950), p. 49 (21st July, 1950), p. 97 (4th Aug., 1950). "North of Scotland Hydro-Electric Schemes: The Loch Sloy Project." *Engineer*, vol. CXC, p. 57 (21st July, 1950), p. 87 (28th July 1950), p. 115 (4th Aug., 1950). "Loch Sloy." *Water Power*, vol. 2, p. 185 (Sept.–Oct. 1950).

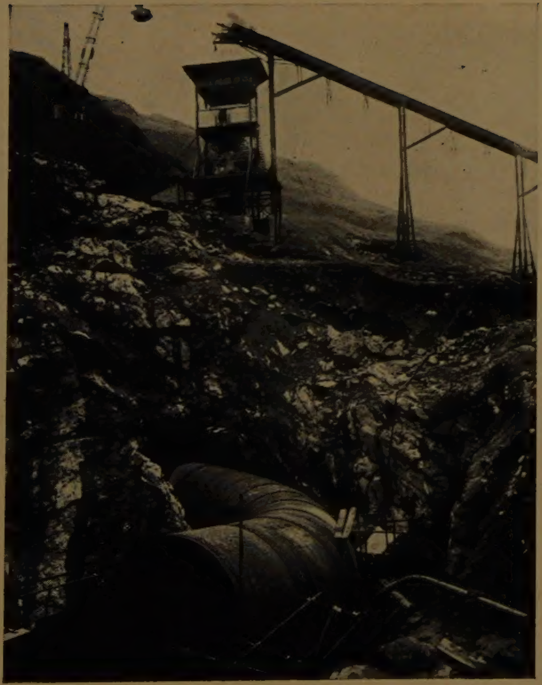
[2] A. A. Fulton, "Loch Sloy Hydro-Electric Scheme." *Chart. Civ. Engr*, Mar. 1951, p. 6. *Also* "Civil Engineering Aspects of Hydro-Electric Development in Scotland." Proc. Instn Civ. Engrs, Pt. I, vol. 1, p. 248 (May 1952).

Fig. 1



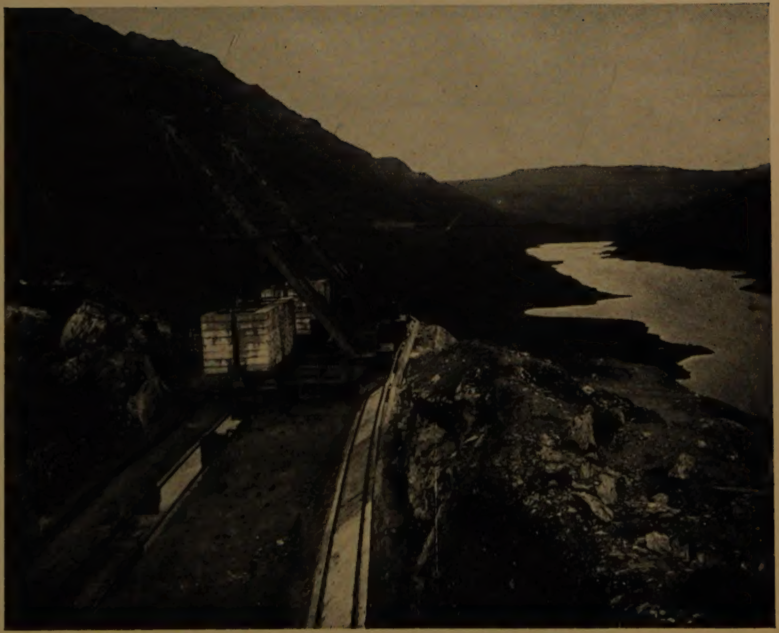
Loch Sloy Dam on the 24th October, 1950

Fig. 4



MOUTH OF INTAKE TUNNEL. BATCHING PLANT IN BACKGROUND

Fig. 7



CABLEWAY TAILMASTS

horizontally to 10 vertically. (See Figs 3, Plate 1.) At 889·68 O.D. the vertical sides of the buttresses commence to corbel out to form circular barrel arches which span between buttresses. The springing of these arches is at right angles to the backslope. The upstream edge of the underside of an arch meets the buttress "T"- head along its elliptical downstream face. Above the arches the dam rises generally in a solid gravity section to 920·00 O.D., and thereafter with curved backslope of 32 feet 6 inches radius to the underside of the roadway at 940·0 O.D.

To enhance the appearance of the structure there are pilaster blocks at the top of the dam at 65-foot centres.

A roadway 11 feet wide, with surface at 941·50 O.D., is provided across the dam and is bounded on both sides by reinforced-concrete parapets which are corbelled out from the dam upstream and downstream faces.

The spillway section is formed by a reinforced-concrete apron spanning between buttresses Nos 5 and 6, and 6 and 7, which gives this section a massive solid appearance although, in fact, under the apron the buttresses and arches are formed similarly to the others. At the top of the dam a bridge carries the roadway over the spillway crest, whilst at the lower end the spillway terminates in a curved concrete bucket submerged in a stilling pool.

A 4-foot-diameter scour pipe is built into the centre buttress of the spillway. At the downstream end of this pipe, and clear of the dam, there is a control valve of the disperser-needle type which is sufficient to regulate the water level of the reservoir when all diversion aqueducts are shut off. When the aqueducts are shut off, the catchment area of the loch is reduced from 32 square miles to 6 square miles. To enable the needle valve to be inspected and maintained, an emergency guard-gate of the Glenfield tube-roller type is provided which operates at the dam face. In front of this gate there is a concrete bellmouth and fixed screen, and downstream of it a transition Meehanite casting changing in section from 5 feet high by 4 feet wide to 4 feet diameter at the pipe. This emergency gate is electrically operated from a small gear house built over the roadway at the top of the dam, the front of the gear house being corbelled out from the dam face to allow the gate to be housed there. To keep the gate steady during hoisting and lowering, cast-iron guides extend from the gate opening to the gear house, the dam wall being plumb over this depth on a 9-foot width.

A chamber in the gravity-dam section near the spillway records the spillway discharge by collecting a flow proportional to that passing over the spillway crest.

The main intake is located on the loch bed near the middle of the dam at the block designated buttress No. 9. The front of the entrance is protected by a fixed coarse screen of 5-inch-by-$\frac{1}{2}$-inch bars at 6-inch centres supported by heavy structural steelwork. To obtain a large effective working area the screen is in conical form between concrete wing walls.

Downstream of this is a finer vertical screen of 3½-inch-by-½-inch bars at 3-inch centres, 25 feet high by 20 feet wide, which can be raised to the intake gate house for cleaning. Behind the lifting screen the intake chamber gradually contracts to an opening 19 feet high by 10 feet wide at the control gates.

There are two gates, one 10 feet upstream of the other. The upstream gate is an emergency guard-gate of the special Glenfield tube-roller type which is not used in normal operation, and is kept in the intake gear house at road level; the other is the main control-gate and this is of the Stoney free-roller type.

Above the intake chamber a tower constructed against the dam face provides access to the gates and the tunnel, acts as an air vent, and houses the gate guides. The intake control house at road level is corbelled out above the intake tower walls, and contains electrically-operated hoists for lifting the fine screen and operating the gates.

Downstream of the control gates there is a transition duct, changing from rectangular section 19 feet high by 10 feet wide to circular section at a 13-foot-6-inch-diameter steel pipe. Encased in concrete, this pipe bends in both the horizontal and vertical planes and then extends into the main tunnel (see *Fig. 4*). The lower half of the transition section is lined with a heavy Meehanite casting and this and the steel pipe protect the concrete from the dangerously high water velocities which, when the gate is " cracked " open for refilling the tunnel, reach about 103 feet per second under the full head of 160 feet.

THE TEMPORARY WORKS

(See *Fig. 5* and Fig. 6, Plate 2.)

Cableways

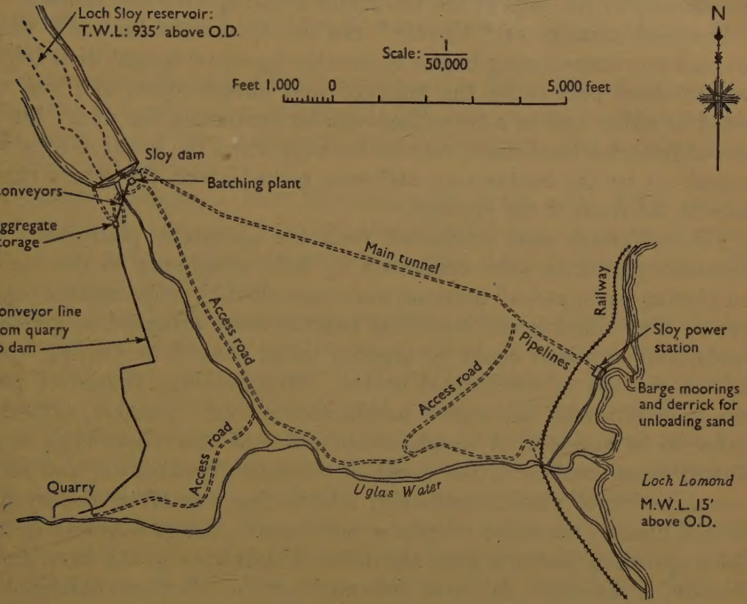
Two Henderson electrically-driven aerial cableways, equipped for hoisting, traversing, and radial travelling, and operating independently, of each other, were used on the construction of the dam. The decision to use cableways was taken by the employers and, under the conditions of the contract, the use of the cableway plant was made available to the contractor free of charge. Erection, operation, and maintenance were, however, the responsibility of the contractor, who also bore the cost of all repairs.

The following particulars may be of interest :—

Maximum load	10 tons
Span	1,350 feet
Height of head-mast	125 feet
Centres of head-mast	12 feet
Height of tail carriages	36 feet
Centres of tail tracks for tail carriages	25 feet
Length of tail carriage tracks	210 feet

Depth of lift of hook	175 feet from below crest of dam
Height of lift of hook	19 feet above crest of dam
Main suspension cable	2¾ inches diameter
Approximate speeds and motor-powers :—	
Hoisting, 250 feet per minute	250 B.H.P.
Traversing, 1,000 feet per minute	
Radial travelling, 25 feet per minute	45 B.H.P.

Fig. 5



Location Plan

The fixed head-masts were of latticed steel construction supported by back and side stay-ropes securely anchored into large blocks of concrete. Since the tail carriages were equipped for radial travelling, a swivel head was mounted on the top of each head-mast, to which the main cables and button-ropes were fitted, together with pulleys for the hoisting- and traversing-ropes. At the base, each mast was mounted on a universal joint and hinge pins carried on a sectional steel grillage secured to a large concrete foundation block.

The radial-travelling tail-masts were sited well clear of the other end of the dam (*Fig. 7*). The front and rear of the masts were each mounted on two ball-pivot bearings each carried on two 4-wheeled bogies running on 3-inch-wide steel rails. The maximum wheel load on one rail wheel was 34 tons. The foundation concrete of these tracks required constant maintenance, owing to the high bearing pressures from the tail carriages—about 180 tons of ballast in the form of concrete blocks and concrete block containing steel punchings as aggregate being carried on each tail carriage. Movement of the carriage was by electric motor geared to four wheels on each bogie at front and rear. These motors were controlled from the operators' platform at winch houses, situated at the other side of the valley beside the head-masts. Power-transmission wires to the motors were supported from ropes which spanned from head-masts to tail-masts. Limit switches were fitted to the tail carriages to prevent them over-running at the ends of the track and colliding with one another.

The load carriage or " bicycle " ran on six main cable-track wheels mounted on compensating bogies to provide for equal weight distribution, and contained pulleys for the hoisting- and button-ropes, and was connected at either end to a travelling-rope for traversing the dam. On the run out from the head-mast, carriers for supporting the hoisting-rope were tripped out by the button-rope and were picked up on the return run by horns at the front of the carriage.

The cableways were controlled from the operators' platforms. The operators' platforms were connected by field telephones to the loading and placing-bays, and all motions were controlled by three master control drums. The hoisting and travelling controls were arranged to give slow speeds, and any load could be lowered under control at varying speeds without the use of mechanical brakes. Overspeeding, failure of power supply, or returning the control handle to the " off " position, caused the brakes to be applied. A simple combination of semaphore-type signals allowed visual control in the event of telephone breakdown, and indeed a combination of visual signals and telephoning was often employed for isolated lifts, to minimize telephone movement. Large windows in front of the operating platform gave the driver a clear view of the dam, and, in addition, a dial-type indicator mounted on the platform indicated the position of the load in both the horizontal and vertical planes.

Owing to the great weight carried by the tail carriages, radial travelling was restricted to a speed of 25 feet per minute to limit the momentum of the carriages and permit instantaneous braking. Since radial travelling over the dam was even slower than this, in operation this movement was reduced to the absolute minimum. In concreting, therefore, with repetition lifts, the cycle of cableway operations consisted of hoisting, traversing, and lowering-out a number of times without radial movement but with varying traversing distances until the concreting bay had been crossed, and then radial travelling upstream or downstream into position for the next concreting run across the buttress. During traversing, the loads were carried fairly high to reduce swinging (of the load) when stopped. The drivers soon became fairly skilled in handling the cableways and would eliminate this swing by stopping traversing momentarily just before the desired position was reached and moving forward again as the load was about to move back from the limit of its swing. If necessary, when lowered to just above the bay, the skip could be pushed by hand easily and quickly over a short distance, to come exactly to the required position. In cases where it was desired to move the load slightly farther upstream or downstream than tail-carriage travel permitted, the load could be winched, for example, by bulldozer, a maximum distance of 30 feet beyond a distance of 350 feet from the head-mast. Over the first 350 feet from the head-mast this offset was limited to 15 feet.

Generally, the cableways proved to be extremely successful during both day- and night-shift operation. Lost time due to breakdown and repairs was less than 2 per cent of hours worked—major repairs being as follows :—

Downstream cableway.—New travelling-rope fitted after 3,000 working hours, that is to say, after handling 71,000 tons ; a new button-rope after 9,000 hours and a new hoist-rope after 5,000 hours, that is, after handling about 100,000 tons.

Upstream cableway.—New travelling-rope fitted after 4,800 working hours, that is to say, after handling about 99,000 tons ; a new button-rope after 11,000 hours and a new hoist-rope after 6,000 hours, that is, after handling about 120,000 tons.

Stretch in each main suspension cable was taken up after about the first 4,000 hours and no further tension was applied. One strand of both main cables failed after approximately 9,000 working hours. The free ends in each case were tucked under adjacent strands and brazed down. This was not entirely successful, however, because the ends so treated pulled out repeatedly until, eventually, the strand was removed along almost the entire length of the cable, no trouble being experienced thereafter.

Three main types of skip, each of 4 cubic yards capacity, were used with the cableways. For transporting excavation material a simple heavy-tray-type of skip was used, carried by a four-legged sling and emptied by unhooking manually two of the end ropes, hoisting and up-ending. For concreting in large spaces a bottom-dump-type of skip was used and carried when full by a six-legged wire-rope sling. Two of the legs were used to keep the bottom closed and dumping carried out by unhooking these ropes and hoisting. For concreting in relatively confined spaces a bottom-dump skip was used with controlled discharge. This skip was divided into two compartments each of 2 cubic yards capacity, discharge from each compartment being controlled by a cross-bar lever.

Batching Plant

One central batching plant of 4 cubic yards capacity was installed containing two Stothert & Pitt 70/54 non-tilting drum-type concrete-mixers. An open storage hopper at the top of the batching plant, divided into six separate compartments each of 20 tons capacity, contained sand and the various grades of coarse aggregate. Sand was lifted to this hopper from a stockpile nearby by a 7-ton “ Monotower ” crane fitted with a

1½-cubic-yard grab. The coarse aggregate was fed to the hopper by a rising conveyor and discharged into the appropriate grading compartment by means of a swivel chute fitted at the conveyor head. Cement was supplied to a separate enclosed hopper by a bucket elevator from the cement shed below.

The sand and the various grades of coarse aggregate were weighed cumulatively in a weighing hopper situated under the main storage hopper.

Cement was weighed separately, in a small enclosed hopper, by a single steelyard connected to a precision indicator which operated over a short range on either side of the correct weight. Routine daily cement consumption checks were carried out to reconcile this weighing with the actual number of cement bags despatched from the shed to the elevator.

Water was measured volumetrically on the control panel by a manometer scale graduated in pounds.

After mixing, the concrete was discharged into a storage hopper, from which 4-cubic-yard batches were drawn off as required into a travelling hopper running on a standard-gauge track, and drawn by a 30-horse-power electric winch up a slight incline to the cableway loading bay, where the concrete was discharged into cableway skips through a side opening in the hopper just above rail level. At the commencement of concreting, this loading bay was sited in buttress No. 11 foundation, as the nature of the ground dictated, and extended to the extreme limits of cableway travel. Work on the buttress was postponed for as long as possible, but by July 1949 it could not be held up any longer ; and since, by that time, buttress No. 10 was well advanced, the space between buttresses Nos 10 and 11 was levelled out and the track moved over between these buttresses. It will be evident that this restricted the length of the loading bay at the upstream side due to the dam-wall concrete of buttress No. 10, and necessitated considerable cableway radial travel in concreting near the upstream face, but unfortunately this could not be avoided.

Sand and Coarse Aggregate Supply

The coarse aggregate, a mica-pyroxene-diorite, was used in 2½"–1½", 1½"–¾", ¾"–$\frac{3}{16}$" gradings. This was quarried about 1½ mile from the dam and transported to stockpiles near the dam site by a series of twelve belt-conveyors. The last of these conveyors from the quarry rose on a steel gantry above the stockpiles, and the stone was discharged from the belt by a travelling tipper on to the appropriate bing. By the Conditions of Contract the employer supplied the Contractor with this stone which the Contractor purchased at this site.

Under the stockpiles, a concrete tunnel had previously been constructed, and stone was drawn off through feed chutes in the roof as required, according to its grading, to a second belt-conveyor system which carried it out of the tunnel to the batching plant. A conveyor attendant in this tunnel was in telecommunication with the batching plant.

This series of conveyors from the stockpiles to the batching plant was made up of three separate belt sections, each discharging on to the next, and each made up of standard 9-foot lengths with inverted troughs carrying the idlers. The belt was 24 inches wide and troughed, and travelling at 241 feet per minute could handle a maximum of 100 tons of aggregate per hour. The three conveyors on this length were driven by 20-horse-power electric motors and had a total length of 1,240 feet and a total vertical lift of 165 feet. The last conveyor was built up on steel trestles and rose 103 feet to the batching plant at an angle of 18 degrees. The conveyors were protected where necessary by corrugated sheeting, bent to semi-circular section to prevent the belts being blown off by ever-recurring high winds.

At the head of No. 2 conveyor a side-discharge chute was fitted from which stone could be loaded to lorries if required.

It was originally schemed to supply sand to the batching plant via the conveyors from a stockpile built up beside the aggregate stockpiles, and loading to the belt as required through the tunnel roof in the same way as the aggregate. This proposal was abandoned in favour of a stockpile situated beside the batching plant, from which sand was lifted by a Monotower crane-and-grab, thereby providing an alternative method of loading stone to the batcher in the event of a conveyor stoppage. Although, generally, the conveyors operated with little trouble, now and again breakdowns did occur, and latterly some trouble was experienced in starting in cold weather with belt slip due to increased roller resistance. During these stoppages, therefore, the Monotower was invaluable in allowing concreting to continue.

Again, when the Monotower was not usable, mainly during periods of high winds, sand was loaded to the batching plant by the conveyors. For this purpose an emergency stockpile was maintained at a convenient point beside the second conveyor section, and sand fed to the belt down a timber ramp, manually and by bulldozer.

The sand was obtained from a natural deposit at Balloch at the southern end of Loch Lomond about 22 miles from the dam site. Those who are familiar with the narrow, winding and busy road along the west bank of Loch Lomond will appreciate the difficulties of making upwards of one hundred return runs daily on this road. To handle the necessary quantities of sand, therefore, barge transport on the Loch was employed. Two tugs of 60 tons displacement were used, one 120-horse-power diesel and one 100-horse-power steam, each drawing three barges and each barge containing 50 tons of sand. To ensure a quick turn-round, fifteen barges were available. The tugs and barges had to be transported overland to the Loch because the River Leven, the natural outlet from Loch Lomond which flows to the River Clyde, is not navigable over part of its length. At Balloch a jetty was constructed for loading the barges. At the other end, at Inveruglas, about 3½ miles from the dam, off-loading was by 6-ton derrick-and-grab to a storage hopper from which the sand was loaded to 5-ton tipping lorries and transported to the stockpiles at the dam site.

Cement

Ordinary "Portland" cement was used throughout, supplied in bags and stored in a shed 150 feet long by 40 feet span by 12 feet to eaves, having a maximum storage capacity of 1,500 tons, which provided at least 2 weeks requirements in storage. Access was by five roller-type doors on the east side through which lorries could enter. The bags were hand-loaded in conjunction with a gravity-roller to a flat-belted conveyor running at a speed of 80 feet per minute along the west side of the shed, and were slit open manually before falling off the end of this conveyor into a cement pit, the bags being retained on a grillage and removed. From here the cement was screw-fed to a bucket-elevator by which it was lifted to the storage hopper at the top of the batching plant.

Water Supply

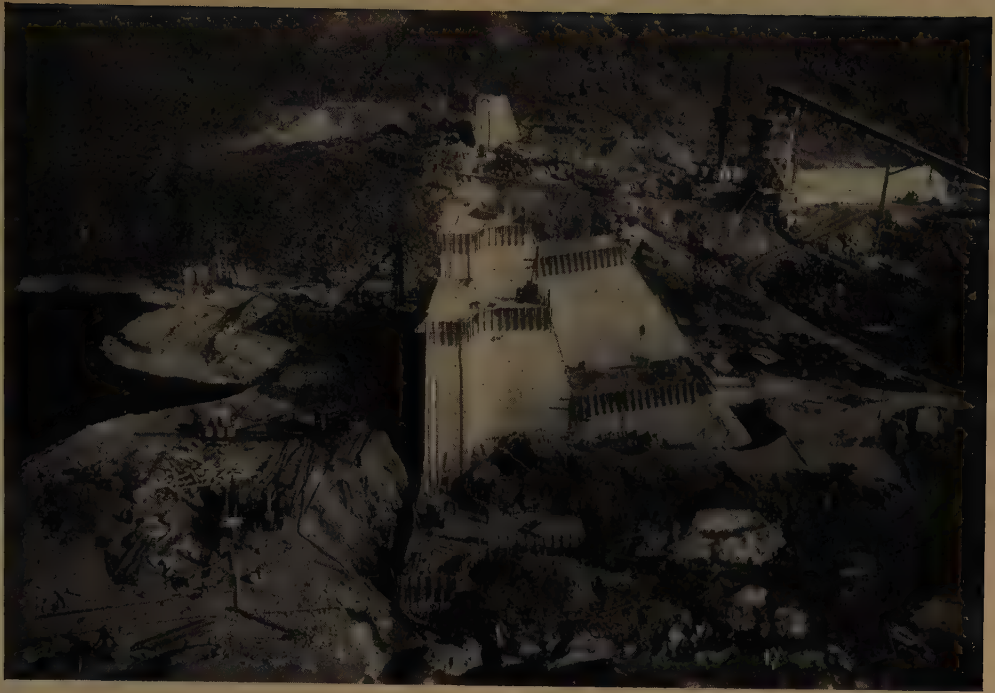
A water-storage tank of 12,800 gallons capacity was installed near the head-masts. This was fed by a hillside stream and supplemented when necessary by pumping from the loch into the water main, which was laid from this storage tank to the other end of the dam on the downstream side, with branch lines (which were extended as work progressed) running up the buttress backslopes. This provided a constant supply of water under pressure for cleaning prior to, and washing off after, concreting. Other branch lines supplied water to the batching plant, garage, and office buildings.

Workshops and Stores Building

Six Nissen huts, 72 feet long by 24 feet span, were used as a garage, stores buildings, joiners' shop, compressor house, electricians' shop, and machine shop.

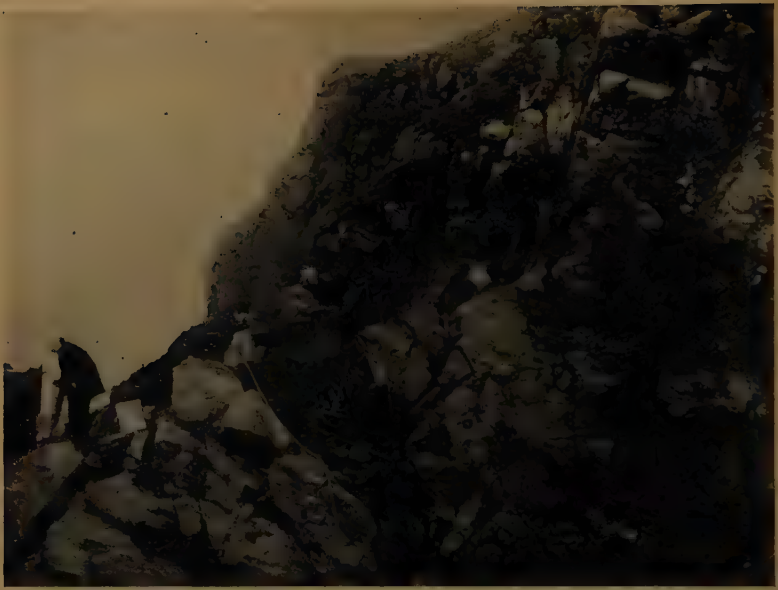
The compressor house contained three stationary compressors of 200 cubic feet per minute capacity each driven by a 55-horse-power electric motor. These were coupled to two portable compressors each of 210 cubic feet per minute capacity, and one portable one of 125 cubic feet per minute situated outside the compressor house. Also in the field were two portable diesel compressors, one of 210 cubic feet per minute capacity and one of 125 cubic feet capacity, sited as required to boost the pressure in the air-mains. The air-line, generally, ran beside the water-line, with similar branches up the back-slopes, etc., to operate scarifyers, jack-hammers, vibrators, and pumps, and to supply air for washing-off and cleaning.

Fig. 8



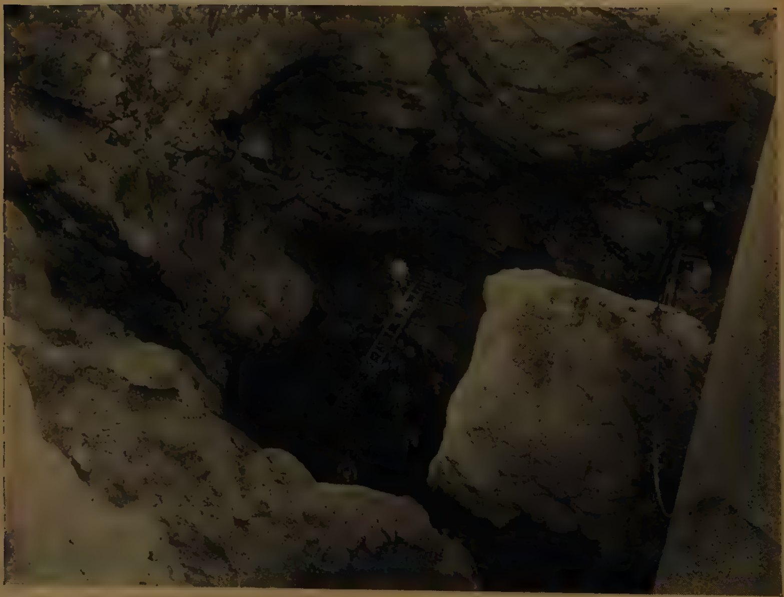
View from West End on the 7th October, 1948. Cofferdams may be seen at the Left

Fig. 11



Excavation of Foundations

Fig. 12



Pocket in Block 17 Cut-off Trench (Bottomed at 85 feet below Original Ground-Level)

Electrical Distribution

Power for the dam construction, the other works, and camps of the scheme was provided by a temporary diesel-electric station of 3,600 kilowatts installed capacity, constructed and operated by The North of Scotland Hydro-Electric Board at Inveruglas. Distribution to the various sites was by overhead 11,000-volt line. At the dam this was led by the Contractor halfway round the site, skirting the cableway winch-house and back anchor, and terminating at the front of the intake.

Four transformers, stepping the voltage down to 440 volts, were installed ; one of 500 kilowatts beside the employer's terminal pole for power to Monotower, batching plant, conveyors, machine shop, and lighting, etc., two of 400 kilowatts (one beside each cableway winch-house for cableway operation, including power for tail-carriage travelling) ; and one of 350 kilowatts at the front of the intake for power to 10-ton and 7-ton derricks at the front of the dam.

The site was suitably floodlit for night-shift work. Lighting cables taken up the buttress backslopes permitted the use of floodlights at working level.

Field telephone cables also were extended up the buttress backslopes as work progressed so that lighting, field telephone, water and air services were always available on each buttress at working level.

Camp and Access Road to the Site of the Dam

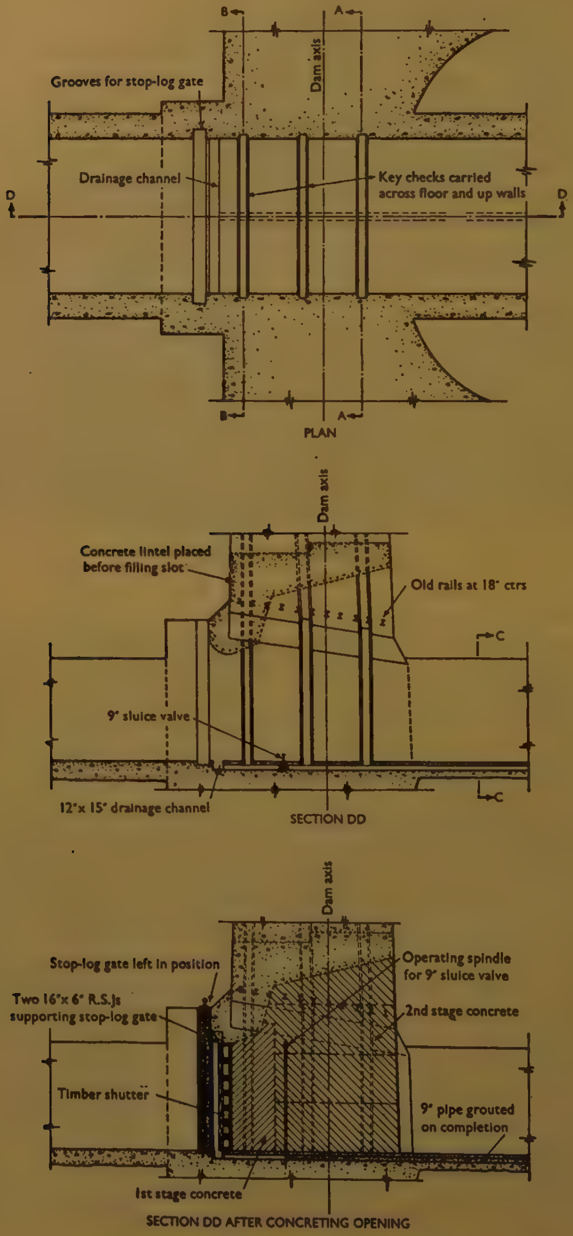
These were built under separate contracts before the construction of the dam was begun.

Cofferdam and Diversion of Loch Outlet

Formation of an earth cofferdam was commenced early in 1947 by bulldozing overburden forward into the loch from foundations Nos 7, 8, and 9. As excavation progressed this cofferdam was extended to protect the foundations in the depression between buttresses Nos 4 and 9, using material removed from the excavations. (*Fig. 8.*)

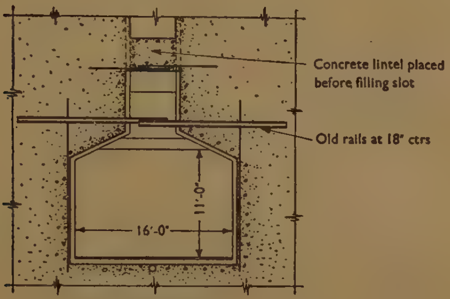
For by-passing the water flowing from Loch Sloy during the construction of the dam, a temporary opening was left through the dam wall between buttresses Nos 7 and 8 and a concrete flume constructed. The Uglas water, the natural outlet from the loch, was allowed to continue flowing until the early summer of 1948 by which time the diversion flume had been concreted. This flume extended upstream and downstream well clear of the excavations and almost through the cofferdam. On its completion, the small remaining portions of cofferdam blocking the ends were cleared away and the Uglas water diverted by bulldozing specially prepared and selected material across the stream on both the upstream and downstream sides, thus completing the cofferdam.

Figs 9 (a)

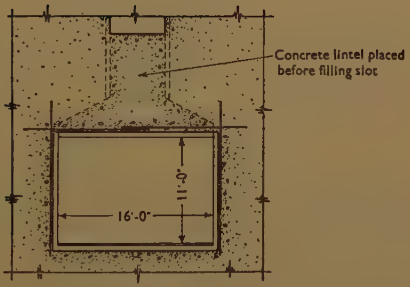

B
A
Dam axis
Grooves for stop-log gate
Drainage channel
Key checks carried across floor and up walls
D
D
B
A
PLAN
Dam axis
Concrete lintel placed before filling slot
Old rails at 18" ctrs
C
9" sluice valve
C
12"x 15" drainage channel
SECTION DD
Dam axis
Stop-log gate left in position
Operating spindle for 9" sluice valve
Two 16"x 6" R.S.Js supporting stop-log gate
2nd stage concrete
Timber shutter
9" pipe grouted on completion
1st stage concrete
SECTION DD AFTER CONCRETING OPENING

DETAILS OF TEMPORARY OPENING

Figs 9 (b)

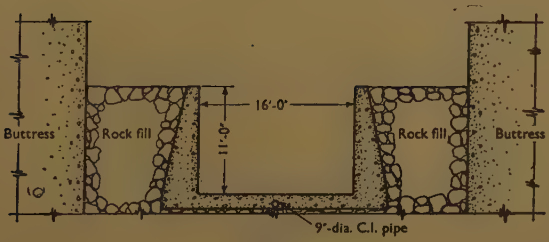


SECTION AA



SECTION BB

Scale: 1 inch = 20 feet



SECTION CC

DETAILS OF TEMPORARY OPENING

A certain amount of leakage did unfortunately occur through the upstream section of cofferdam here, and this was somewhat troublesome when excavating the deeper parts of the cut-off trench for buttress No. 6. To deal with this, sheet-piles were driven along the stretch of cofferdam through which the leakage was occurring and then a deep sump was excavated behind this curtain to intercept and collect the flow still passing. Pumping was carried out continuously here and in a second sump in the cut-off trench of contraction slot No. 5/6, until the cut-off and foundations of buttresses Nos 5 and 6 were concreted.

The temporary opening is illustrated in *Figs 9* (*a*) and (*b*), from which it will be seen that above the flume the buttresses corbelled back to normal contraction slot dimensions. (Contraction slots were left between buttresses. They were 6 feet wide at the front and tapered to 4 feet wide at the rear and were left open until the bulk of buttress concrete contraction had taken place.) The reinforced-concrete lintel above the opening here was not concreted until just prior to filling the contraction slot. It will be seen that grooves for a stop-log gate were formed in the walls of the flume upstream of the opening; and a channel across the flume invert just downstream of the gate position drained to a 9-inch cast-iron pipe embedded in the flume base, and allowed leakage water from the stoplog gate to run-off, so preventing any build-up of water pressure on the newly-placed concrete when plugging the opening. This pipe was grouted on completion.

By February 1950, work on the dam and the rest of the scheme had progressed sufficiently to allow impounding to commence. This first stage flooding was to 840·00 O.D., which is 60·00 feet above the original loch level and 95 feet below the permanent spillway crest. At this 840·00 O.D. level an emergency spillway was left in contraction slot No. 7/8 above the diversion channel, to deal with any possible sudden rise in the loch, and a concrete apron was also formed at the sides of the diversion channel below to prevent scouring by water falling from the emergency spillway.

Concreting of the temporary opening was carried out in two stages. A wall, 4 feet thick, was first placed in one 12-foot-high lift across the upstream portion of the opening and then later the downstream portion was concreted in 5-foot-high lifts, the 9-inch sluice valve on the cast-iron pipe not being closed till after the second of these 5-foot lifts had been placed.

The stop-log gate was designed to withstand the full head to the emergency spillway, to ensure that water pressure would not be transmitted to the newly-placed concrete of the plug because of failure of the gate. The gate, which consisted of 12-inch-by-12-inch timbers bolted together, was supported at one-third span by two vertical R.S.Js. The upstream face of the first stage wall was shuttered clear of these R.S.Js. Timber shuttering was used and on the upstream side this was naturally concreted in. At regular intervals 2-inch-diameter steel pipes were left pointing into the key-checks on the roof of the opening and through these pressure grouting was subsequently carried out.

The whole of the first-stage operation was completed in a few hours. During this time a D.8 bulldozer levelled down the final strip of cofferdam in front of the intake, blocked the upstream end of the diversion flume with earth from the cofferdam and travelled out to the downstream side of the dam through the diversion channel. The R.S.J. bearers for the stoplog gate were then erected and the stoplog gate lowered into position by the cableway. The prefabricated timber shutters were then set up, the necessary reinforcements placed, and concreting carried out to the full height of the opening.

Later, in July 1950, the emergency spillway slot at 840·00 O.D. was concreted, and impounding carried out to 893·00 O.D. A similar emergency spillway had been left at this level in contraction slot No. 7/8 and this was left open until the remainder of the permanent works had been completed to top water level.

EXCAVATION

About 36,000 cubic yards of peat and sandy loam material overlying rock, and about 56,000 cubic yards of rock were excavated for the dam foundations and cut-off trench. The rock was a mica schist throughout and although heavily fissured and badly weathered in places, especially near the surface, and with one major fault at the course of an old waterfall and stream bed at foundations Nos 15, 16, and 17, the soundness of the rock generally improved fairly rapidly with depth of excavation to a hard blue mica schist which forms a satisfactory foundation for the dam. The bottom of the cut-off assumed from the original foundation investigation borings and the actual bottom cut-off levels reached are shown in *Fig. 10.*

The material overlying rock in the foundations was excavated in open cut with side slopes cut back to an approximate 1 : 1 batter.

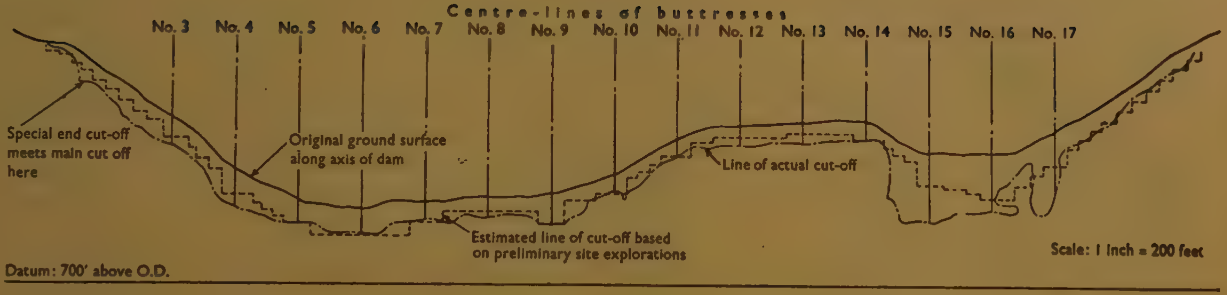
Generally, excavation was carried out by D.8 bulldozer, Lorain $1\frac{1}{2}$-cubic-yard dragline, 24 R.B. face shovel, and 2/10 Smith excavator, loading to tippers and dumpers, cableway and derrick-trays, as best suited the particular foundation. Much of the rock excavation had to be loaded to skips manually owing to the difficulty of moving mechanical excavators in the latter stages of the work, and of course all the final trimming was by hand. (*Fig. 11*, facing p. 179.)

Initially three derrick-cranes were erected on rails, one 10-ton at the intake, a 7-ton on the cofferdam, and a second 7-ton at foundation No. 15.

By March 1948, when the temporary works had been constructed, and were ready for mass-concreting in large quantities, the bulk of the excavation had been carried out from foundations Nos 7 to 14.

In foundations Nos 10, 11, and 12, however, final cleaning and washing down later revealed some fairly extensive fissures and sand seams

Fig. 10


Centre-lines of buttresses
No. 3
No. 4
No. 5
No. 6
No. 7
No. 8
No. 9
No. 10
No. 11
No. 12
No. 13
No. 14
No. 15
No. 16
No. 17
Special end cut-off meets main cut off here
Original ground surface along axis of dam
Line of actual cut-off
Estimated line of cut-off based on preliminary site explorations
Scale: 1 inch = 200 feet
Datum: 700′ above O.D.

ANTICIPATED AND ACTUAL FOUNDATION LINE OF CUT-OFF TRENCH which had to be followed down and cleaned out. Similar fissures were discovered in many of the other foundations later, and in most instances were not apparent until final clearing of the rock had been carried out. As all these fissures had to be cleared out by hand it was a lengthy operation, especially when this cleaning out would reveal more and more fissured rock which, of course, was all removed before concreting could commence. Such fissures naturally caused considerable delays to concreting.

At foundation No. 15 excavation proceeded intermittently by hand (according to the labour available) in a small area at the downstream side of the foundations to locate the bedrock. Some fairly sound rock was encountered and sealed by a lift of concrete in December 1947, at level 808·00 O.D. Upstream of this it was apparent that the rock dipped to considerably lower levels and that, in fact, a major fault existed here; and so it was decided to postpone further excavation until diamond drill boring investigations had been carried-out. Six bores were driven and these indicated that sound rock for a suitable cut-off would not be reached till at least 780·00 O.D. It was decided, therefore, to excavate over the entire area between buttresses Nos 15 and 16, removing the material between buttresses as well as the material in the normal foundations, and then concrete to ground level over this area between the sound rock which existed on either side of foundations Nos 15 and 16, but founding on a poorer quality of rock between these limits than would normally have sufficed.

In all about 26,000 cubic yards of excavation was carried out here, mainly during the summer months of 1948, by Smith 2/10 excavator and Lorain 1½ cubic yard dragline, and by October 1948 concreting had commenced in the downstream part of the excavation. At this stage it was decided to allow concreting to continue forward over the upstream area of the foundation above the cut-off, before the cut-off excavation had been carried out, and to excavate later in tunnel for this cut-off by driving a heading from a shaft in the contraction slot No. 15/16. This enabled concreting to commence considerably sooner than would otherwise have been possible, especially as heavy timbering or trimming of the front slope to ground level would have been necessary for the cut-off trench excavation.

The bottom of this cut-off was at level 763·00 O.D. which was 75 feet below original ground level.

A 10-ton derrick was erected in March 1949, immediately downstream of buttress No. 16, to cover the zone not commanded by the cableways. This did much useful work removing spoil, and concreting the awkward foundations in this area.

In the spring of 1949, a heading was driven from the bottom of the shaft, at level 773·00 O.D., under the region of foundation No. 16 cut-off, to investigate the rock at this level. This was very heavily fissured for 30 feet and when it improved in quality the heading was stopped at 54 feet. Excavation of the fissured rock was then carried out from the surface to this tunnel, and concreting carried out in the usual way after concreting the portion of the tunnel remaining in sound rock.

On the cut-off trench of foundation No. 17 an old waterfall pot-hole was unearthed and cleared of sand and pebbles to its bottom at 771·00 O.D. (85 feet below original ground level), where sound rock was encountered. (*Fig. 12.*)

At the hillsides, generally, excavation was carried out for the lower foundation in the first instance and the next higher foundation not commenced until the previous buttress was concreted to a sufficiently high level, spoil from the higher foundation being disposed of against the next lower buttress.

A heavily fissured and badly weathered seam of rock running across the downstream portion of foundations Nos 1 and 2, which obviously would have required major excavation to clear out entirely, was discovered early in 1950. Owing to the high level of these foundations it was decided to avoid major excavation here and merely concrete solidly against the whole area of the rock on the downstream face of the excavation, commencing the normal back slope at ground level. This fissure joined another large sand-filled fissure at foundation No. 1A on the normal cut-off trench and then ran into a large mass of soft material to the west. Investigations were carried out by rock-drilling and trial-pits to determine the most suitable route for an end cut-off. From the information gained, a shaft was sunk 55 feet downstream from foundation No. 1A on a line at 45 degrees to the dam axis, with the intention of driving a heading along the sand seam on this line to the dam concrete; but rock here proved to be more heavily fissured and badly weathered than was anticipated, so a complete cut-off trench was excavated from the shaft to the concrete of foundation No. 1A, penetrating all this fissured rock, and completely cutting the sand seam. A heading was driven through some poor rock on the other side of the shaft to complete the seal.

PRESSURE GROUTING THE FOUNDATIONS

Pressure grouting was carried out along the line of the cut-off trench to ensure a complete seal to the passage of water. This was done after the buttresses were about two-thirds completed, by injecting grout through holes spaced at approximately 7-foot-6-inch centres and less in fissured rock. These holes were first grouted to depths of 60 feet and 30 feet alternately. In cases where grout was taken readily the depth was increased first in the 30-foot holes and then, if required, in the 60-foot holes. Thus the greatest depth drilled was 120 feet in the poorer quality rock, and the least 30 feet in sound rock. For some time, grouting holes through the cut-off trench concrete were formed using 4-inch-diameter steel pipes as cores, which were pulled up as concreting progressed. Forming grout-holes through concrete in this way was, however, later abandoned and the holes drilled in the normal way through the concrete, because it was found difficult to continue the pipe alignment precisely through successive concrete lifts, difficult to extract the pipes, and in addition the pipes and their supports caused an obstruction and consequent delay to concreting; moreover if a pipe was accidentally slightly displaced when concreting the hole was rendered useless.

Generally, at fissures, numerous 4-inch-diameter and 2-inch-diameter-mild-steel pipes were concreted into the fissure and pressure grouting later carried out through the 2-inch pipes, and rock drilling and subsequent grouting carried out through the 4-inch pipes.

The grouting procedure was as follows. When the buttresses were about two-thirds completed the first 20 feet of rock was drilled and then grouting commenced using a thin mix of about 20 per cent cement, 80 per cent water, designed to travel as far as possible into the rock cracks and fissures. A build-up of pressure would indicate that the grouting was being successful and would be stopped some time after the pressure had reached 1½ to 2 times the static-water pressure of the dam. If the pressure, however, did not begin to build up, say, after 1 ton was injected without diminution of rate, the mix would be gradually thickened.

After the initial grouting the holes were flushed out and then tested after 24 hours, and if the test was successful, drilling was then carried out for a further 10 feet, and the whole procedure repeated to such a depth that no further grout could be injected.

The normal hydraulic-pressure test was not used as this gives no indication of water lost in the test. An air test was substituted. Air pressure from the air main at 100 lb. per square inch was applied for 20 minutes to a stand-pipe sealed into the hole, and if after that time the water level had fallen more than 6 inches in a 4-inch-diameter pipe or 2 feet in a 2-inch-diameter pipe, the test was not considered successful and the hole would be reinjected and the cycle of operations repeated until the test was passed.

Altogether 16,000 linear feet of drilling was entailed and 460 tons of cement injected as grout. This drilling and grouting was carried out under sub-contract.

CONCRETE

Five main classes of concrete were used in the permanent work and were designated according to the cement content of the mix and the maximum-size grading of the coarse aggregate. The mixes were specified as 3, 4, 5, 6 and 7 cwt of cement each to 12 cubic feet of dry sand and 20 cubic feet of crushed stone, but after tests the proportions were adjusted to approximately 3¼, 4, 5, 6 and 7 cwt of cement to 10·2 cubic feet of dry sand and 22 cubic feet of stone.

The second figure in the mix designation denoted the maximum size of the coarse aggregate, thus a 4/2½ mix was one specified as 4 cwt of cement to 12 cubic feet of sand to 20 cubic feet of stone and with 2½-inch maximum stone grading; and a 5/1½ mix was one specified as 5 cwt of cement to 12 cubic feet of sand up to 20 cubic feet of stone, but with 1½-inch maximum stone grading. In all large mass-concrete sections 3¼/2½ was used; 4/2½ and 4/1½ in smaller mass-concrete sections requiring slightly greater strength, for example, in retaining-walls and surrounding the 13-foot-6-inch-diameter steel pipe; 5/2½ in the upstream face of the dam (for a thickness of 2 feet 6 inches), in the cut-off trench and in foundation concrete placed against rock surfaces; 5/1½ in reinforced parts of fairly large section such as the intake tower; 6/¾ in reinforced roof slabs, bridge piers, etc., and 7/¾ in heavily-reinforced pre-cast beam units.

At the commencement of concreting, workability of the mixes was poor, mainly due to the flaky nature of the grano-pyroxene-diorite aggregate and a coarse sand grading. This was greatly improved by increasing the percentage of fines in the sand to about 31 per cent passing B.S. sieve No. 52, and increasing the proportion of coarse aggregate to a mix of about 10·2 cubic feet of dry sand to 22·0 cubic feet of stone. *Figs 13* (*a*) and (*b*) show typical sand and coarse aggregate grading curves.

All mixes, when placed, were vibrated by air-operated "Poker" immersion-type vibrators.

Table 1 shows the 2-cubic-yard-batch proportions as weighed-out in the batching plant, the average slump, and the specified and actual strengths obtained, etc.

Generally two works test cubes were made from each of the mixes used during each day. One of these was crushed at 7 days and one at 28 days, when the concrete was required to show an increase in strength of 50 per cent over the minimum specified compressive strength at 7 days, and also an increased strength over that shown by the cube tested at 7 days. The concrete-testing was carried out at a laboratory on the site. With 2½-inch maximum size aggregate, 8-inch test cubes were used, and with ¾-inch and 1½-inch maximum aggregate, 6-inch test cubes were used.

During the period of concreting there were two serious 7-day-cube failures; one in September 1948 gave only 750 lb. per square inch, and another taken during the following month gave only 900 lb. per square inch. In each case cubes were immediately cut from the lifts concreted with these pours and, at 14 days, crushed at 2,500 lb. per square inch and 2,400 lb. per square inch, respectively. The original 28-day test cubes crushed at 1,340 and 2,030 lb. per square inch.

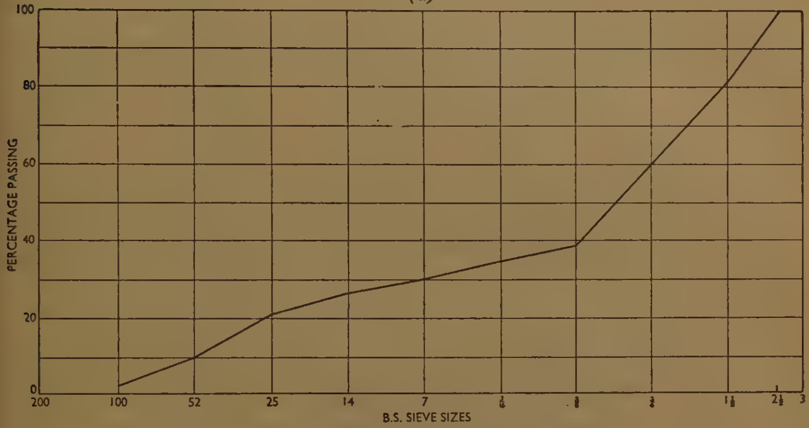
These cube failures were attributed to faulty curing, the cubes having been left exposed in the open and consequently under very different internal temperatures from the mass-concrete of the dam. After this, test cubes were cured thermostatically and no further failures resulted.

Figs 14 show a typical time-temperature graph in the middle of a buttress.

No concreting of the permanent works was carried out when the temperature of the atmosphere or the material was lower than 38° F. on a falling thermometer or 34° F. on a rising thermometer, apart from some mass pre-cast concrete units which were concreted using heated water. For this a boiler was erected beside the batching plant and the heated water discharged into the gauging tank. Heated water was also used when concreting near the specified lower-temperature limits.

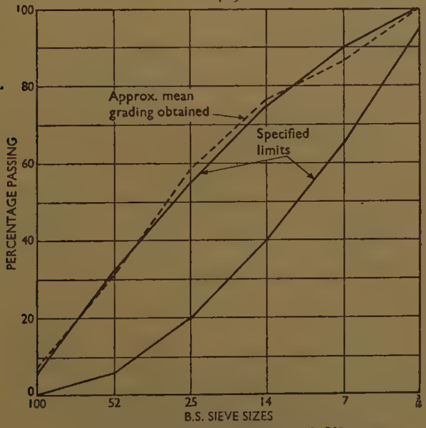
Figs 13

(a)



TYPICAL COMBINED AGGREGATE GRADING CURVE

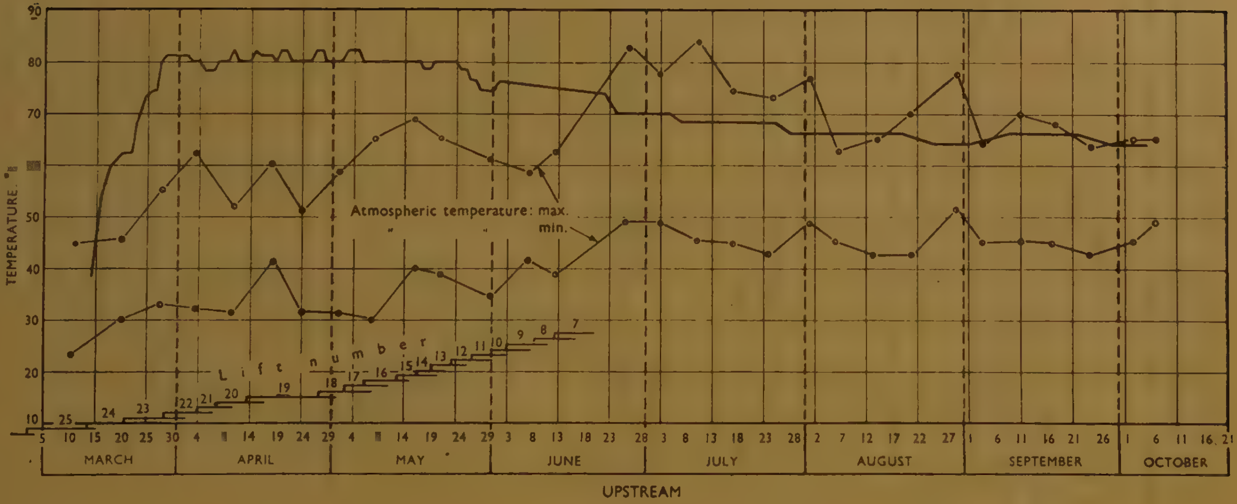
(b)

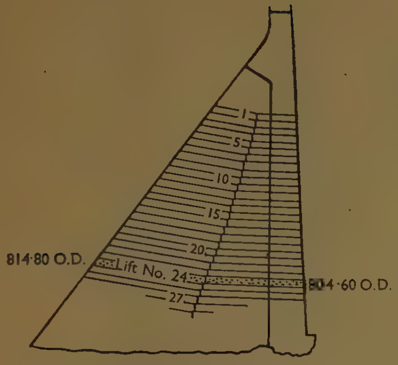


Note.- Sand merchant requested to supply 30% passing B.S. sieve No.52 and remainder within specified limits

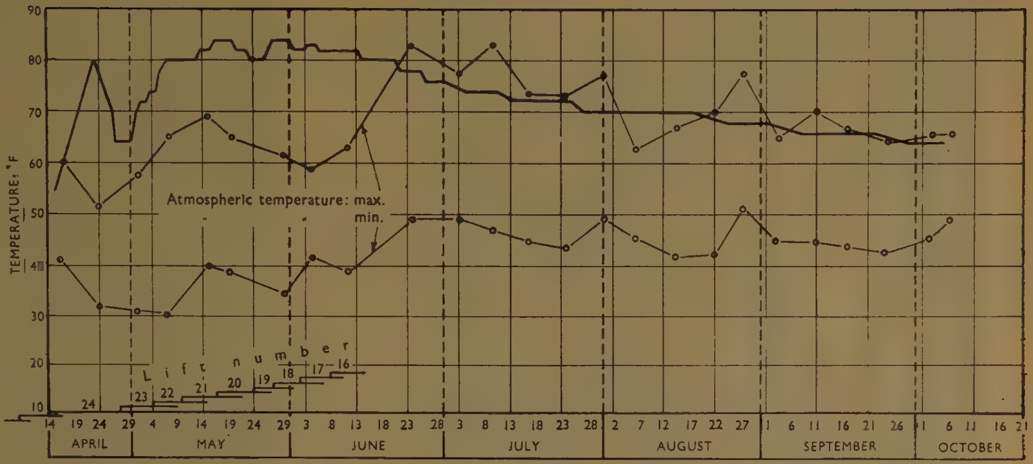
SAND GRADING CURVES

Figs 14


TEMPERATURE. °F
90
80
70
60
50
40
30
20
10
5 10 15 20 25 30
4 9 14 19 24 29
4 9 14 19 24 29
3 8 13 18 23 28
3 8 13 18 23 28
2 7 12 17 22 27
1 6 11 16 21 26
1 6 11 16 21
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
UPSTREAM
Atmospheric temperature: max.
" " min.
Lift number
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7



SECTION SHOWING LOCATION OF LIFT NO. 24



DOWNSTREAM

Time-Temperature Graphs inside Buttress

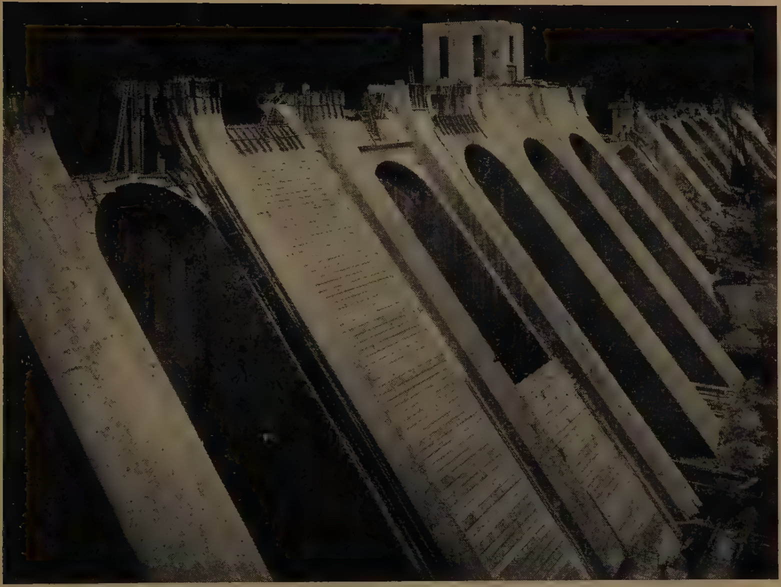
TABLE I.—CONCRETE MIX STATISTICS

Nominal mix	3, 4, 5, 6 or 7 cwt of cement to 12 cubic feet of dry sand, to 20 cubic feet of crushed stone
Modified mix . . . . (Adjusted for workability)	3¼, 4, 5, 6 or 7 cwt of cement to 10·2 cubic feet of dry sand, to 22 cubic feet of crushed stone
Aggregates	Sand.—105 lb. per cubic foot, dry. Moisture content about 9 per cent when added. Specific gravity 2·62
	Stone.—Mica-pyroxene-diorite. Density 176 lb. per cubic foot solid. Specific gravity 2·85
	Dry bulk density 97 lb. per cubic foot. 2 per cent moisture content when added

Class of mix	Total quantity: cu. yds	Batch proportions (yield approx. 2 cubic yards)					Water/cement ratio	Average slump: in.	Density: lb. per cubic foot. (average)	Minimum permissible compressive strength: lb. per square inch		Actual average compressive strength of test cubes: lb. per square inch	
		Ordinary Portland cement: lb.	Sand: lb. (9 per cent moisture)	Crushed stone: lb. (2 per cent moisture)									
				2½″/1½″	1½″/¾″	¾″/3/16″				7 days	28 days	7 days	28 days
3¼/2½ (1/2·5/5·3)	156,000	825	2,600	1,650	1,750	1,500	0·70	Nil	155½	1,300	1,950	1,850	2,626
4/2½ (1/2·0/4·4)	3,000	1,000	2,600	1,650	1,750	1,500	0·60	½	155½	1,650	2,475	1,825	2,420
5/2½ 5/1½ } (1/1·6/3·5)	46,000	1,250	2,600	1,650 —	1,750 2,600	1,500 2,300	0·51	¾	155½	1,900	2,850	2,900	3,820
6/¾ (1/1·3/2·9)	1,000	1,500	2,600	—	—	4,900	0·48	1½	155	2,100	3,150	3,305	4,672
7/¾ (1/1·2/2·5)	2,000	1,750	2,600	—	—	4,900	0·45	2	154½	2,400	3,600	3,274	4,934

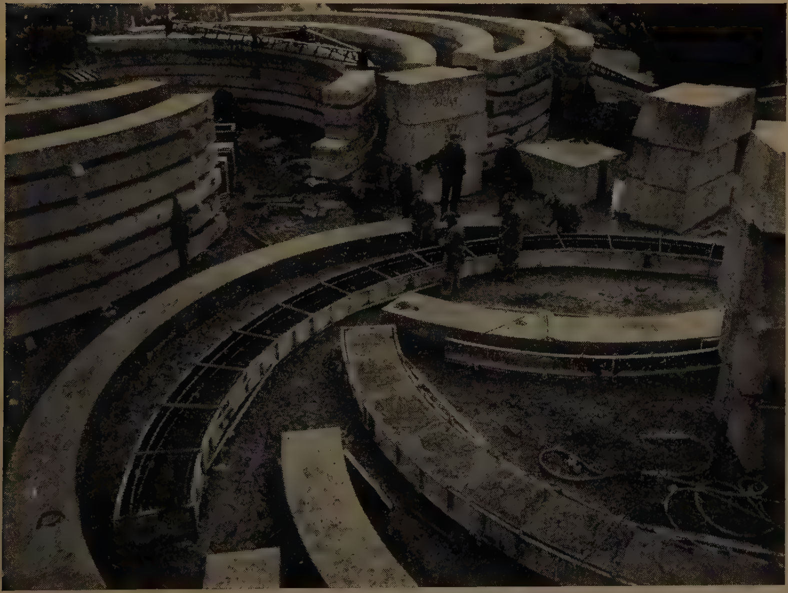
Total: 208,000

Fig. 16



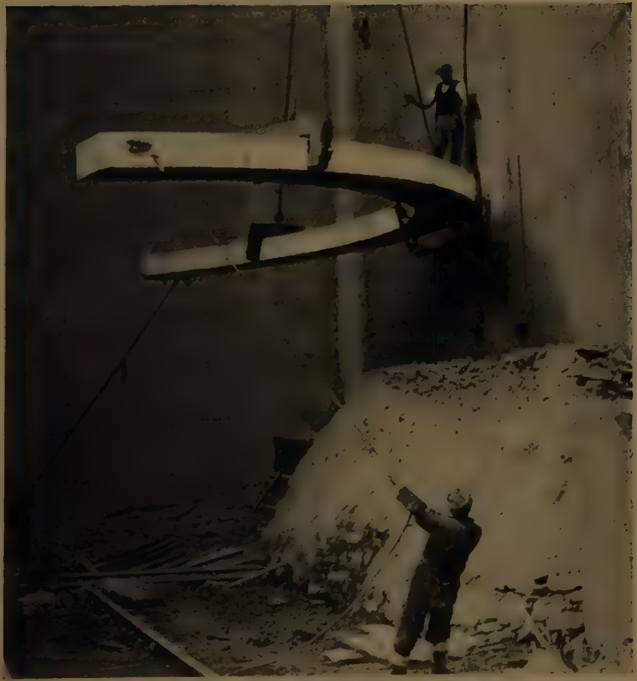
Spillway Section from Right Bank on the 7th May, 1950

Fig. 19



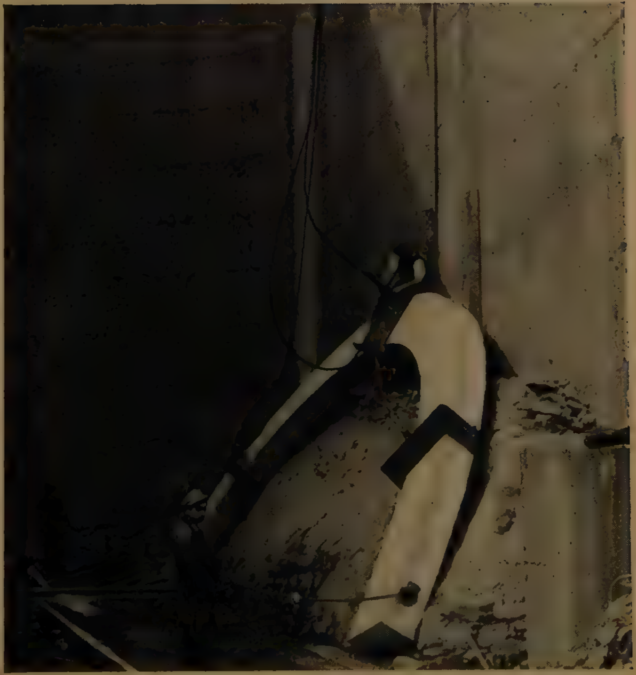
The Pre-casting Yard

Fig. 20



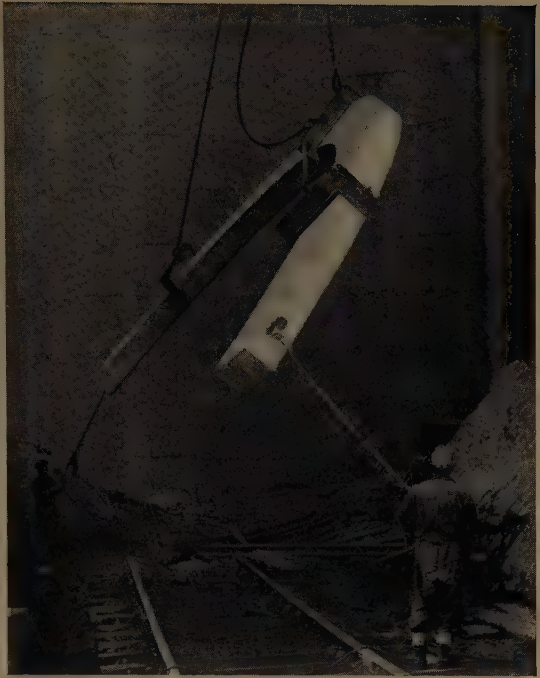
Pre-cast Arch Ring being Lowered by Cableway onto Loading Bank

Fig. 21



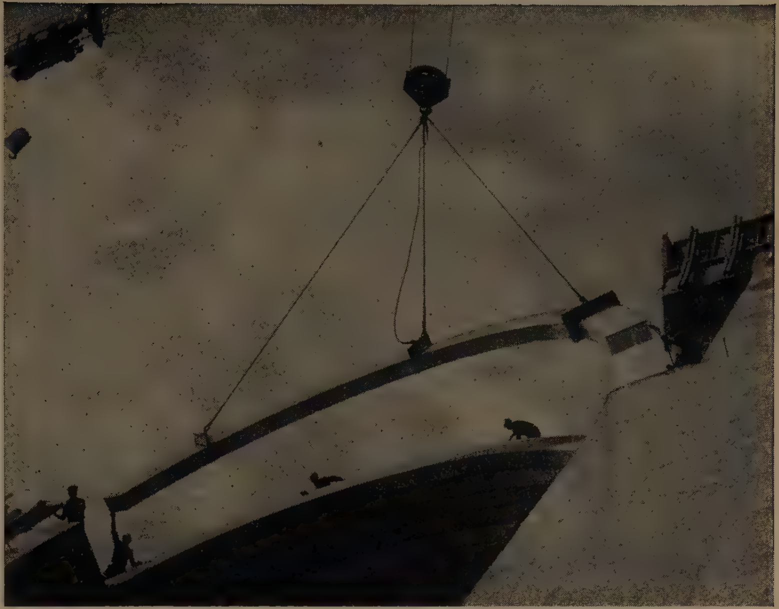
Changing Slings on Pre-cast Arch Ring to give Correct Angle for Placing

Fig. 22



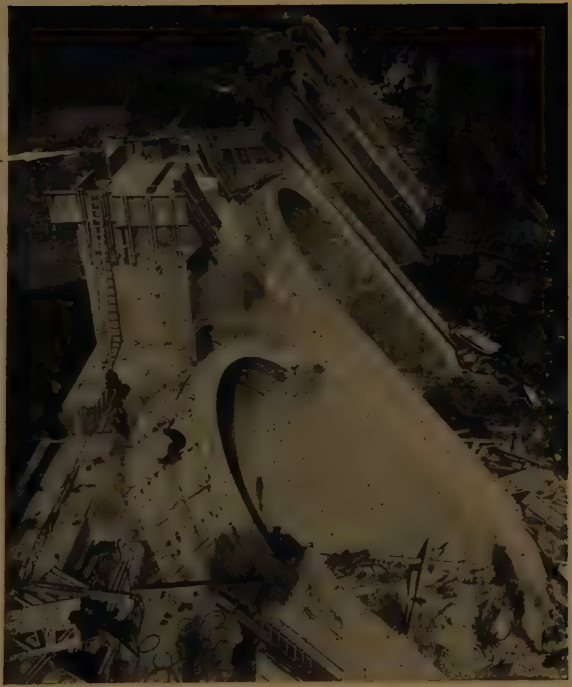
Pre-cast Arch Ring being Hoisted at Placing Angle

Fig. 23



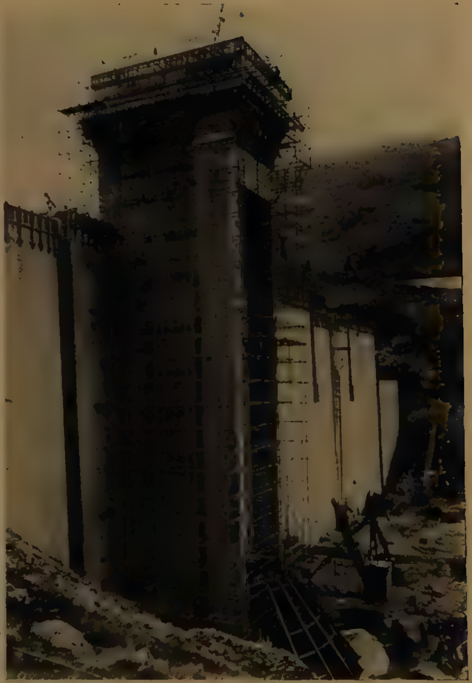
Pre-cast Arch Ring being Placed

Fig. 24



Arch Rings in Position over Blocks 11–12

Fig. 27



Intake Tower under Construction

All concreting was carried out in the dry. Before depositing against rock or concrete surfaces the surfaces were thoroughly washed, wetted, and coated with a layer of stiff neat-cement grout, the grout being deposited immediately in front of the main mass of concrete.

A few hours after concreting, all laitance was washed from the top of the lift by a powerful air-and-water jet and all key-moulds removed. After this washing had been carried out no further work, such as picking or scabbling the surface, was required; all that was required was a second washing just before placing the next lift of concrete. All vertical joints between concrete bays were scabbled using a small rotary air-operated scarifying tool with rip-bits.

Although the use of plums up to a maximum size of 4 tons would have been permitted in the mass-concrete, they were not used because this would have interrupted the concreting procedure. Also, such plums would have interfered with the movement of the shutter cranes.

The dam face below level 882·00 O.D. and other concrete surfaces at the intake and in the intake tower which will be in contact with water under pressure were painted with " Bowranite " shortly before impounding.

PRE-CAST WORK

General Details

During 1949, it became increasingly obvious with the continuous extremely adverse weather and the acute skilled-labour shortage that complicated shuttering at the higher levels would be unwarrantably slow, resulting in a considerable slowing-up of concreting. There was also the question of the arches between buttresses, which had been designed as Voussoir block arches, each arch containing eighty-one Voussoir blocks, each of which would require to be cast on the ground and lifted into position, temporarily supported on centering. Complete designs were evolved for this centering in structural steelwork and also a scheme to use the centering later to support the spillway apron shuttering, which presented another problem.

It was realized, however, that large-scale pre-casting of units weighing up to the maximum cableway capacity of 10 tons was the answer to these various problems, allowing this difficult work to be carried-out at ground level under more sheltered conditions and by semi-skilled labour before being built into the dam.

Altogether the following units were pre-cast :—

1. *Arch rings.*—Each arch span between buttresses was formed by thirteen pre-cast reinforced-concrete arch rings 13 inches thick by 2 feet 6 inches deep, spanning 36 feet 4 inches, with $\frac{3}{4}$-inch clearance between rings which was pointed after placing. The first 16 inches of the span on each side was taken up by abutments which corbelled out from the sides of the buttresses

and sloped at right angles to the buttress backslope. This corbel was cast in situ with the buttress concrete in a special shutter.

The arch rings each weighed 10·2 tons and contained 5·1 cwt of reinforcement.

2. *Spillway beams* (see Figs 15, Plate 2, and *Fig. 16*).—The 39-foot clear span of the spillway apron was formed by L-shaped beams 47 feet long which, with 3-inch clearance at either end from the spillway guide-walls, allowed 3 feet 3 inches bearing at one side and 4 feet 9 inches at the other. 1⅛-inch-diameter dowel bars embedded in each of the buttresses here were bonded to the beams by in-situ concrete poured into recesses provided at the beam ends, as shown in Figs 15, Plate 2. A $\frac{3}{16}$-inch clearance was provided between beams to ensure that no load would be transmitted from one beam to the beam below except at the buttresses. This space was wedge-shaped and expanded-metal reinforcement was placed between beams when pointing on completion.

   Each beam was cast with a camber of 1 inch in the plane of the spillway and ¾ inch in the plane at right angles to this.

   The spillway beams each weighed 10·3 tons and contained 12 cwt of reinforcement.

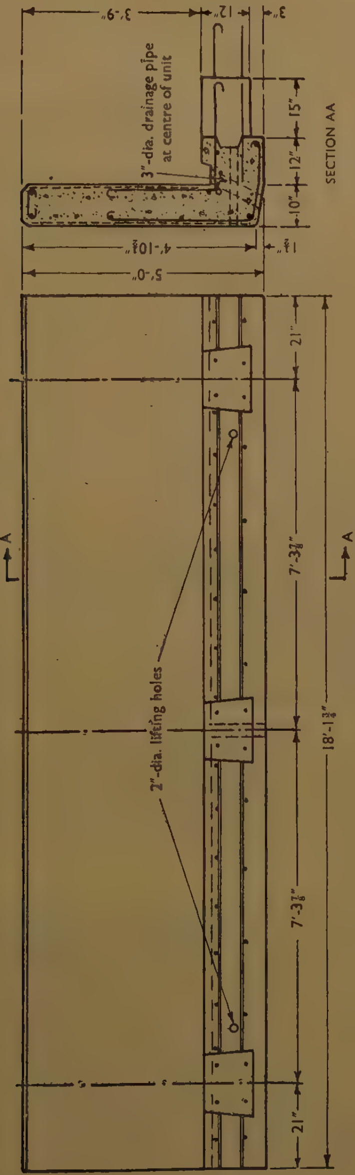
3. *Pilaster blocks.*—The standard pilasters were pre-cast in six large mass-concrete blocks. The most labour-saving of these blocks were the first one, which corbelled out from the upstream and downstream faces, and the top block, which, being narrower and shuttered on all four sides, required more shuttering per cubic yard. The lower blocks weighed about 9 tons each and the top block 5½ tons.

4. *Parapet wall.*—The parapets are 10 inches thick and 3 feet 6 inches high above road surface, and at 18 inches below the road surface corbel 10 inches so that the inside face of the parapet is plumb above the face of the dam on both the upstream and downstream sides. This corbel would have complicated the in-situ casting of the parapets, which would have involved much shuttering at the top of the dam for a small quantity of concrete.

   Sections as shown in *Figs 17* were pre-cast in lengths of about 18 feet. They were supported when placed, by soldiers and by bolting the base-nibs to the dam concrete until the final 18 inches of roadway concrete had been poured and the nibs and projecting steel concreted into the roadway. At the pilaster ends the parapets had a projecting key which fitted into a recess on the side of the top pilaster block. Gaps 18 inches wide were left between one parapet section and the next and

Figs 17

Scale: ¼ inch = 1 foot



Pre-cast Parapet

reinforcement was left projecting from the parapets into these gaps. These gaps were concreted in situ, using a special steel shutter clamped to the parapet.

The parapet sections as pre-cast weighed 7·2 tons and contained 3 cwt of reinforcement.

Because of the lack of suitable casting and storage space, and the many other commitments for the labour available on the site, it was decided to have the parapet sections cast off-site. This was done by a well-known pre-casting firm near Glasgow and the units were transported to the site by lorry.

All the other sections, however, were pre-cast at the dam site.

5. *Spillway bridges.*—The main girders were pre-cast as shown in *Figs 18.* These had dowel bars left projecting to tie the in-situ placed deck-concrete to the girders.

   The deck-concrete was poured on "inverted T"-shaped reinforced-concrete crossbeams which were also pre-cast and which, spanning between girders, formed a permanent shuttering. The pre-cast girders weighed 9½ tons and contained 14 cwt of reinforcement.

The Pre-casting Yard (see *Fig. 19*, facing p. 192)

The area behind the cement-shed was levelled out by bulldozer to form a space for the pre-casting yard, and the 10-ton derrick-crane was removed from the front of the intake during August 1949, when the intake had passed out of its reach, and erected at the pre-casting yard.

From here, a standard-gauge rail track was laid on a slight incline to a point within the cableway-reach between buttresses Nos 9 and 10. A winch was installed at the pre-casting yard end of the track and a carriage of two bogies was used to transfer the pre-cast units to the cableways. In the case of the arch rings a special articulated platform, fabricated on site, was used to support the rings and restrict bending stresses during transit.

Arch rings were stacked six deep and were each separated by six concrete blocks which had to be accurately set to level before placing the next ring, since the rings had little bending strength when lying flat just after lifting from the casting base.

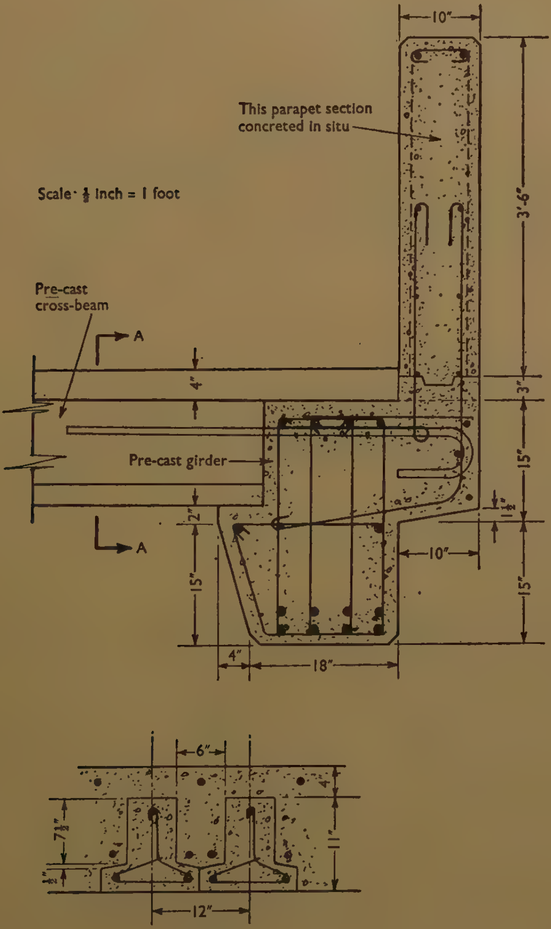
The spillway beams were too unstable to be stacked more than two deep. The pilaster blocks were easily stacked one on top of the other and, since they were lighter than both the arch rings and the spillway beams, they could be cast and stacked at a greater radius from the 10-ton crane, and in any case were easily handled and transported to other stacking places if required.

In November 1949, a subsidiary spillway-beam casting-yard was established in the space between buttresses Nos 8 and 9. The beams cast here were stored between other buttresses.

Moulds and Casting Bases

Casting was carried out in steel-sided moulds which bolted to specially prepared concrete bases. The bases were poured within the moulds and finished with a smooth surface which was kept well greased and oiled.

Figs 18



SECTION AA

SPILLWAY BRIDGES

The mould sides thus overlapped the concrete bases, to which they were held tight by bolts which passed from side to side through steel pipes embedded in the base concrete. The tops of the moulds were tied by flat tie-bars above the level of the casting.

In general, four beds were laid for each set of shutters; thus, if a shutter was used every second day (this was normal practice), by the time the fourth casting was made the first had sufficient strength to be lifted from the casting to the stacking base.

In the case of the spillway beams, the steel moulds formed only part of the shuttering and served to hold in position timber boxes which formed any departures from rectangular section. These timber insertion pieces were much more difficult to strip than the steel and received a good deal of rough handling and damage. As they could not be stripped until the beams were lifted for stacking (when about 7 days old), four sets of boxes were required for each set of steel moulds.

The arch rings and beams lay at least 6 days on the casting bases before being lifted in normal weather and at least 7 days in cold weather. On the stacking bases they were then left to mature for 28 days before being further handled for placing.

Lifting and Placing

The arch rings were lifted after either 6 or 7 days by a triangular latticed strongback, which gave a vertical lift to three T-pieces which lay under the ring and flush with the casting-bed surface.

After 28 days they could be lifted by a three-legged wire-rope sling attached to lifting bridles held to the rings by 1½-inch-diameter steel pins. An additional shorter leg was attached to this sling which, when fixed to the bridle at the crown, held the ring at the required slope for placing on the dam. The other longer leg attached at the crown lifted the ring horizontally.

The rings were transported from the blockyard in a horizontal position and lifted off the transfer carriage by the cableway. About 10 feet from the ground the centre bridle was attached to a hook fixed in the side of the adjacent buttress and the ring lowered slowly, so that the two ends fell and the centre leg slacked off until the shorter, centre leg could be fixed to the centre bridle. With the ring thus held at the correct angle it was lifted a little and the bridle released from the hook on the buttress; the ring was then hoisted up to be placed in position. (See *Figs 20* to *24* inclusive, between pp. 192 and 193.)

The spillway beams were lifted from the casting bases by a long two-legged wire-rope sling, the ropes passing under the beams through holes left in the casting bases. Timber packing prevented the concrete surfaces from being damaged. For subsequent lifting at 28 days, for placing in position, the beams were lifted by a sling attached to two eyebolts which passed through holes in the beam. These holes were inclined along the beam to align with the angle of the sling legs and across the beam to ensure that when lifted the beam lay at the correct slope for placing.

The pilaster blocks were lifted by a lifting ring attached to the blocks by three eyebolts screwed into nuts embedded 18 inches in the concrete from the top of the block.

The bridge girders were handled merely by casting two steel hoops into the top of the units in the vertical plane of the C.G.

The parapets were hoisted into position on the dam by a four-legged sling attached to the casting by a steel bar which passed through holes provided in the base.

The cableway proved to be invaluable in placing these various pre-cast sections quickly and accurately. By taking most of the weight of the units on the cableway they could be pushed and crowbarred, without much difficulty, into their precise positions.

Altogether 174 arch rings, 148 spillway beams, and 152 pilaster blocks along with the 8 bridge girders and the cross beams were pre-cast on site. Casting commenced during August 1949, and by the end of October of the same year, the first arch span (No. 8/9) was erected in position. By Easter Day 1950, all the units had been pre-cast. Almost all the parapet sections were placed in position on the dam in 2 weeks.

In all, 3,140 cubic yards of concrete was pre-cast.

Welding of Reinforcement

Most of the reinforcement for the pre-cast units had to be found from steel already on site, which had been ordered and delivered in the requisite lengths for the in-situ cast sections. Some of this had already been bent so it was decided that, to utilize as much as possible, these various lengths should be welded to suit the new pre-cast sections. As some of the pre-cast units were very heavily reinforced, with little room for bar laps, this welding greatly simplified the re-shaping of the steel and saved much time in building it up into the various frames which were, of course, merely lifted by crane into the pre-casting moulds when required.

The Secrom process was chosen for the welding because it requires no elaborate preparation of the bar ends, no special adjunct to form weld-reinforcement, and does not require rotation of the bars during welding. The bars to be welded are held together in a special copper mould, in strict alignment end-to-end by means of cotter pins, and welded. Welding was by direct current, $1\frac{1}{8}$-inch-diameter bar welds taking 12 minutes to complete, and $\frac{7}{8}$-inch-diameter bars 7 to 8 minutes. This process has been fully described in an earlier Paper.[1]

Construction Details Including Notes on Shuttering

After the cut-off trench and foundation concrete had been poured, concreting of the buttress above commenced. To allow longitudinal contraction to take place along the dam wall, contraction slots were provided between buttress sections as already mentioned, and to deal with lateral contraction the upstream section of a buttress was concreted 1 month ahead of the corresponding downstream section. A transverse joint, dividing the buttresses into these sections, took the form of vertical steps rising 3 in 1, in 3-foot-8-inch steps, to meet the furthest upstream point on the arch springing at 889·68 O.D. To this level, the upstream portions of buttresses between contraction slots were concreted without complication.

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[1] E. J. Buckton and J. Cuerel, "The New Waterloo Bridge." J. Instn Civ. Engrs, vol. 20, p. 145 (June 1943), *Figs 17*, p. 162.

Rigid conventional-type steel shutters with soldiers, as shown in Figs 25, Plate 2, were used. A 4-foot depth was adopted for these shutters which, with a 4-inch overlap on the bottom concrete, gave a standard 3-foot-8-inch lift. Special steel elliptical shutters were provided for the elliptical section made up in two 11-foot-8-inch lengths.

To allow the contraction slot and buttress side-shutters to bolt flush with these elliptical shutters, which were rising on a 40 in 1 batter, the top of the upstream section lifts were constructed rising towards the downstream side at right angles to this, that is to say, at 1 in 40.

A stout steel " lip " was provided bolted to the top of all shutters to give a clean joint-line. Care was taken to have all joint-lines continuous and uniform throughout, and consequently all lifts were set up accurately to level. This was important also to ensure that level 889·68 O.D., where the form of construction changed, would be at the top of a lift, also to ensure correct lift levels at the underside of pilaster blocks, bearing in mind that although upward adjustment to the extent of about 3 inches per lift was possible, there was very little downward adjustment, since the shutters were set up sitting on the " staff " bolts.

The former-bolts, leaving 1-inch-diameter nuts in the concrete for the " staff " bolts, were tapered from 1¼-inch-diameter to 1⅛-inch-diameter and were held level and in position by a " keep " attached to the soldiers. Being well greased, no ferrules were required. The " staff " bolts were 1-inch-diameter and, with a welded collar and cranked end, could be tightened without a spanner. 4-inch-square plate-washers were used.

Steel-framed cradles were hooked to the top of, and suspended over the outside of, the shutters to allow access for removing and re-fixing " staff " bolts, and for aligning the shutters, adjustment being obtained from timber wedges at the lower end of the soldiers.

The following cranes were used for shutter lifting—

One Jones K.L.44,	capacity	1¾ ton at 14	feet	radius
One Jones Super 40,	„	3 tons at 9	„	„
Five Jones Super 22,	„	2 tons at 8	„	„
Two Neals 1-ton				

These cranes were lifted from buttress to buttress by the cableways as required. Now and again, to speed shutter-lifting, two cranes would be sited side by side on a buttress and a whole side of shutters lifted in one section. The weight of standard shutters with soldiers attached worked out at about 1½ cwt per foot-run. There was a maximum of eight shutter-lifting gangs, each provided with a portable crane and consisting of three men and a crane driver.

To return to buttress construction details: the downstream sections followed the upstream but lagging behind corresponding lifts by at least 28 days. For structural reasons, the top of these downstream lifts was formed sloping at 1 in 6 rising towards the rear. The portable cranes could just negotiate this slope under their own power. There was no complication from here to 889·68 O.D., although it may be of interest to mention that the backslope shutters were set to the correct batter by the shutter gang, using a steel template with plumb-bob attachment. The tops of the shutters were then checked by instrument and any slight adjustment necessary carried out before concreting. The back-slope shutters were similar to the standard vertical shutters, but naturally deeper.

At the upstream section, above 889·68 O.D., a seating had to be formed to support the upstream side of the first arch ring and this was produced by inserting timber shutters (set up parallel to the backslope) against the elliptical shutters which were still used until their area had been completely blanked-off by the timber inserts. The true shape of the edge thus left was semi-circular, matching the semi-circular barrel arches.

The stepped transverse joint was continued, now rising 10 in 8, between these arch seatings and extended over the centre 18 feet of the buttress for the first three lifts, and then stepped out and extended over a 31-foot width to 907·49 O.D.

The downstream section above 889·68 O.D. incorporated the arch springing. The corbels which formed abutments for the arch rings were constructed by timber shutters with steel stiffeners set on top of the standard side shutters and on a special side shutter, all these shutters being left in position until the arch corbels had been completed. Figs 26, Plate 3, clarify this.

For the first corbelled lift, shutters B_1 and C were set up, B_1 being adequately tied across the top from side-to-side and back to the transverse joint, and for the next and subsequent corbelled lifts B_2 was added. It will be obvious that the setting of these shutters was rendered very simple by leaving shutters A in position and by having them set precisely to line and level. Removable panels on shutters B_1 and B_2 allowed fixing to be carried out from the inside.

Above the abutments, the buttress width was normally stepped back to 18 feet to allow concreting to continue before placing the arches, and this increased to 31 feet at the transverse joint which had been stepped out to that width.

Above 907·49 O.D., the contraction slots were changed to two smaller slots set 15 feet 6 inches from the centre line of the buttresses, because otherwise the arches would have been split by these slots. The upstream section between the upper contraction slots was concreted together with the downstream section, the 1 in 6 rise of which was gradually levelled out to give a level concrete joint at the underside of the first two pilaster blocks. At level 920·00 O.D. the straight backslope shuttering was replaced by a curved shutter of similar construction. The 18-foot-wide downstream portion was here increased to 31 feet in width by corbelling above the arch spaces towards the backslope and meeting it along the joint-line at level 920·00 O.D. Thereafter on this section, apart from pilaster block placing, there was no complication to underside of parapet level.

After the completion of arch abutments on adjacent buttresses, arch rings were normally placed without delay and concrete poured in the haunches, and by the time the lower contraction slot under the arch rings was filled (this lagged 3 months behind adjacent-buttress concreting) and reinforcement above the arch rings placed, concreting above the rings continued to the underside of the roadway. The upper contraction slots formed at the sides of this section were filled 2 months after this concrete above the arches had been placed.

In cases where the arch rings were placed immediately after the abutments had been formed, concreting of the buttress downstream section was carried out against the arch rings up to 907·49 O.D. (upstream). The lift above this level was constructed out to the upper contraction slots at the upstream side and returned into the arch rings on the downstream portion; and the next and following lifts carried through at the downstream side to the backslope to the full width of 15 feet 6 inches from the buttress centre-line. This prevented the occurrence of a vertical joint right on top of the arch rings. This form of construction eliminated the complication of an overhang at 920·00 O.D.

At the spillway, the sequence of operations was essentially the same as at standard buttresses. On buttress span No. 5/6 the spillway beams were placed complete to the top, and the crest section then concreted above the arches, although on span No. 6/7 the crest was concreted before all the beams had been placed by corbelling above the position of the top beam. This allowed work on the crest and spillway bridge here to go ahead sooner than would otherwise have been possible.

The spillway nose overhang was constructed by using special bracket soldiers welded-up from standard soldiers on site.

At the intake, the tunnel length of the 13-foot-6-inch-diameter steel pipe was erected at the tunnel portal, and as sections were completed the pipe was winched forward into the tunnel. Rollers attached to the underside of the pipe ran on rails which had been set in concrete in the tunnel. Outside the tunnel the pipe was erected on concrete pedestals, and ultimately completely embedded in concrete here and surrounded by packed concrete inside the tunnel. This concrete was later pressure-grouted through holes provided in the pipe.

The steel pipe consisted of ⅝-inch-thick plates riveted in 8-foot strakes formed in two semi-circles, with triple-riveted covers in boiler fashion on the longitudinal seams and with double-riveted outside cover-straps on the circumferential seams. The bend was made by welded mitres kept clear of the field joints so that the circumferential covers occurred on parallel pipe.

The heavy Meehanite castings at the intake base were accurately set-up and concreted-in as the work progressed. The emergency gate and removable-screen guides, which extended up the intake tower, were built in later.

"Hand-over-fist"-type climbing steel shutters were used on the intake tower, fabricated to the various required shapes. They were 3 feet 8 inches deep and were provided with wedge-shaped striking-sections. They were bolted at the base to the lower shutters which were in turn held to the wall concrete by bolts which did not pass from side-to-side through the walls, but were held by plate washers to the wall concrete. Cardboard ferrules were used on these bolts. Originally, soldiers were not used but they were introduced later to aid in aligning the shutters.

When the tower extended beyond the reach of the 10-ton derrick, shutter-lifting was carried out by K.L.44, fitted with a 40-foot jib. This was sited on pile sections spanning the downstream shaft, and the shutters which these piles fouled were lifted by cableway once the crane had been removed.

Shutter-lifting, and later gate-guide erection and painting, were assisted by tubular scaffolding round the tower and up the shafts although at times the scaffolding sub-contractors had difficulty, due to the weather, in keeping pace with the tower progress when it was under way, and two lifts a week were being concreted. This scaffolding was of particular assistance in setting up the overhanging shutters at the top of the tower. (See *Fig. 27*, facing p. 193.)

The intake house was straightforward. The tower shutters were adopted for it and used in a single row, and with a 2-inch overlap on the concrete, gave a 3-foot-6-inch lift. The roof was cast in situ. It is interesting to record that the windows were first glazed with a reinforced glass set in putty, and within less than a week a considerable number of these panes had been blown in. This was remedied by reinforcing the steel sashes with steel bars and re-glazing using a patent steel beading.

The draw-off tower, being for the most part merely a plumb face at the front of buttress No. 6 with gate guides, and thus being dependent on buttress No. 6 progress, was not completed by the first partial impounding date, and so to enable the emergency guard-gate to be operated then, the lifting gear was temporarily housed on a platform supported by R.S.Js cantilevered from the dam face, and sited above both stage 1 and stage 2 impounding levels. The dam face was recessed at these girders which were later burnt off and the pockets made good.

At the hillsides (excepting foundation difficulties and section No. 2 which accommodated the spillway recorder-well), the gravity-dam sections were extremely simple. They were divided by parallel-sided contraction slots 5 feet wide (with key checks at about 6-foot centres) running straight through from front to rear. Owing to the relatively high level of these sections and the consequent relatively small volumes involved, the upstream and downstream portions were concreted in one.

The total length of steel shuttering used was about 3,600 feet, excluding pre-cast mould shutters.

CONCRETING PROGRESS

The quantity of concrete placed per week varied according to the weather, as would be expected, although slightly out of phase with it since inclement weather had a greater adverse effect on shutter-lifting than on actual concreting.

Broadly speaking, concreting increased from about 600 cubic yards placed per week in the spring of 1948, when mass-concreting commenced, to a maximum of 4,082 cubic yards per week during the summer of the following year. After this, with smaller volumes in lifts, progress decreased to about 2,000 cubic yards per week during spring 1950, and by the end of June 1950, having dropped suddenly to 200 cubic yards per week, it can be said that mass-concreting had by then been completed.

The best daily average concreting-rate worked out at 58 cubic yards per concreting-hour and the best weekly average rate at 45 cubic yards per concreting-hour. These rates were achieved in the fairly early stages when lifts contained large volumes.

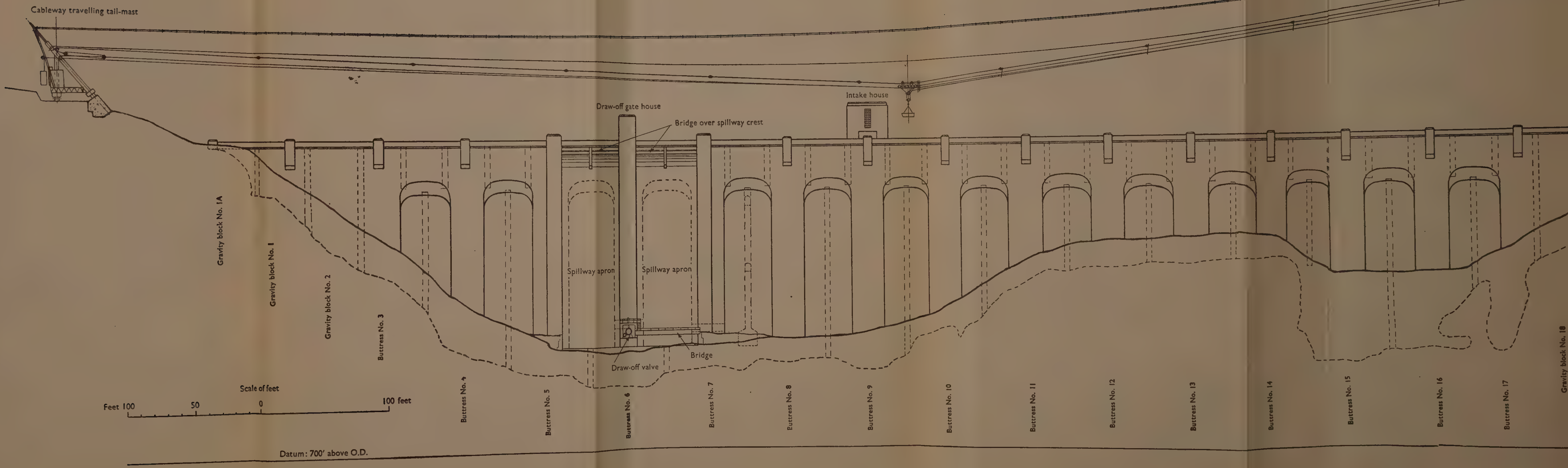
Fig. 28, Plate 3, shows graphically the weekly rainfall and various weekly concrete-placing statistics for the year 1949, and Figs 29, Plate 4, illustrate, pictorially, general progress achieved during 6 monthly periods.

From the autumn of 1948, two 10-hour shifts were worked and concreting was generally carried out during both shifts until early 1950. After this, most of the concreting was carried-out by the nightshift, leaving the dayshift to concentrate on placing pre-cast units, crane lifting, etc., because at this stage, with so much radial travelling, only one cableway could be used to its full efficiency at one time except in special cases.

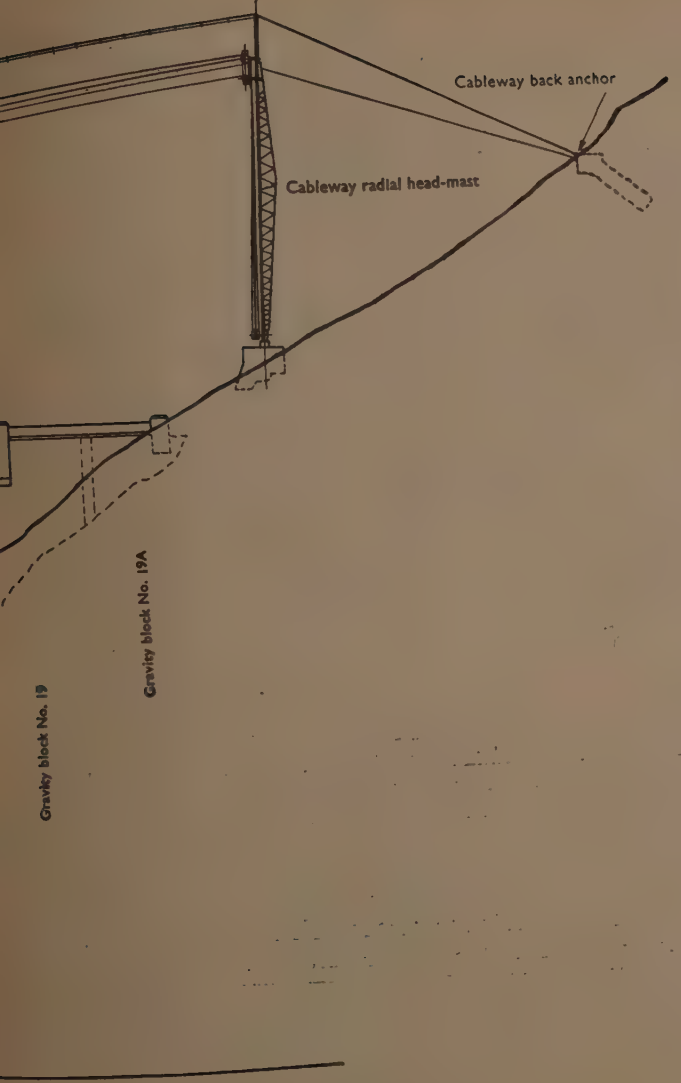
In all, 208,000 cubic yards of concrete, 48,000 tons of cement, 70,000 square yards of shuttering, 3,140 cubic yards of pre-cast concrete and 300 tons of reinforcement were entailed in the permanent works.

Fig. 1 shows the dam in October 1950.

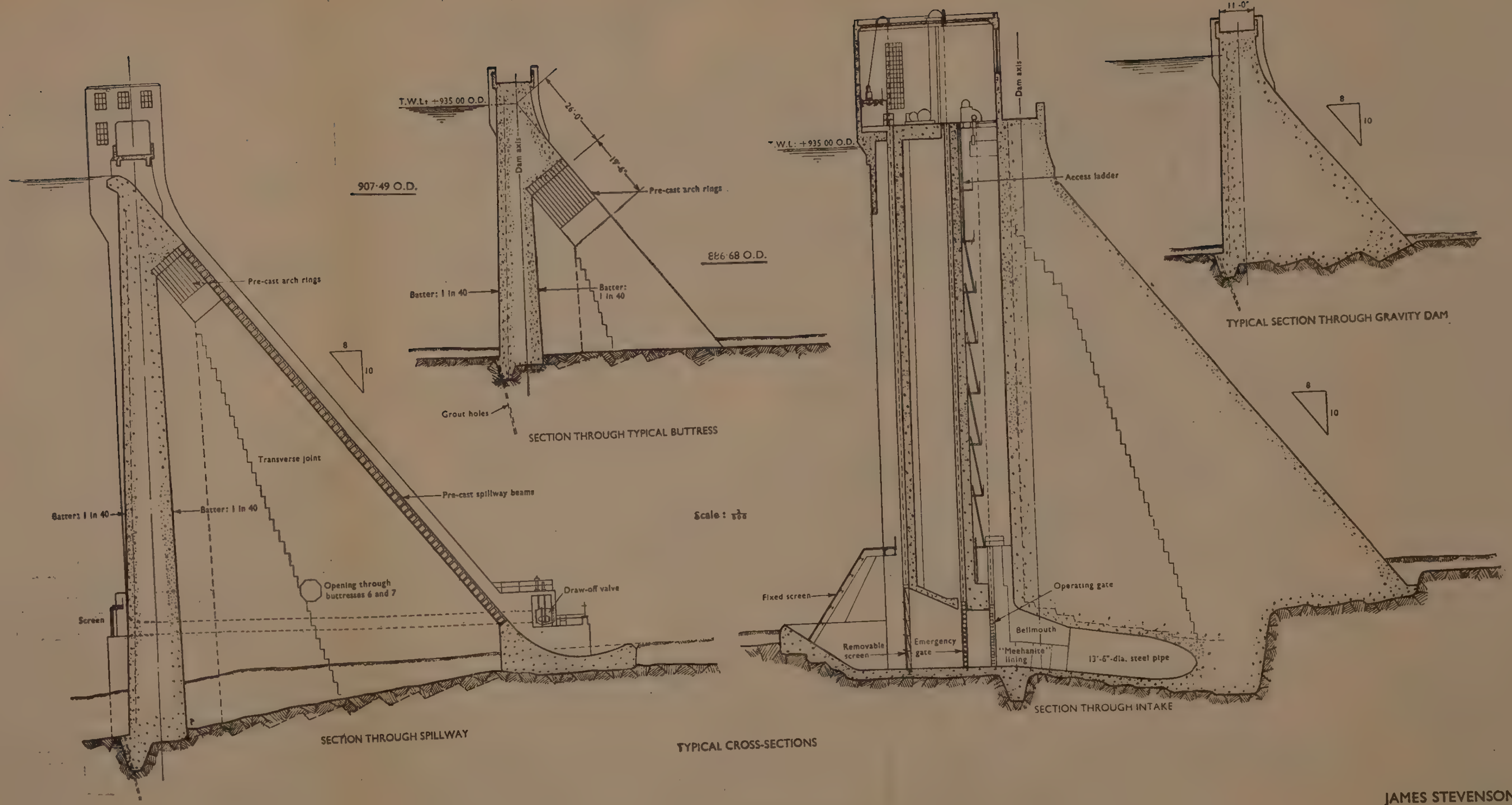
FIG. 2



ELEVATION OF DAM FROM DOWNSTREAM SIDE



FIGS 3



SECTION THROUGH SPILLWAY

SECTION THROUGH TYPICAL BUTTRESS

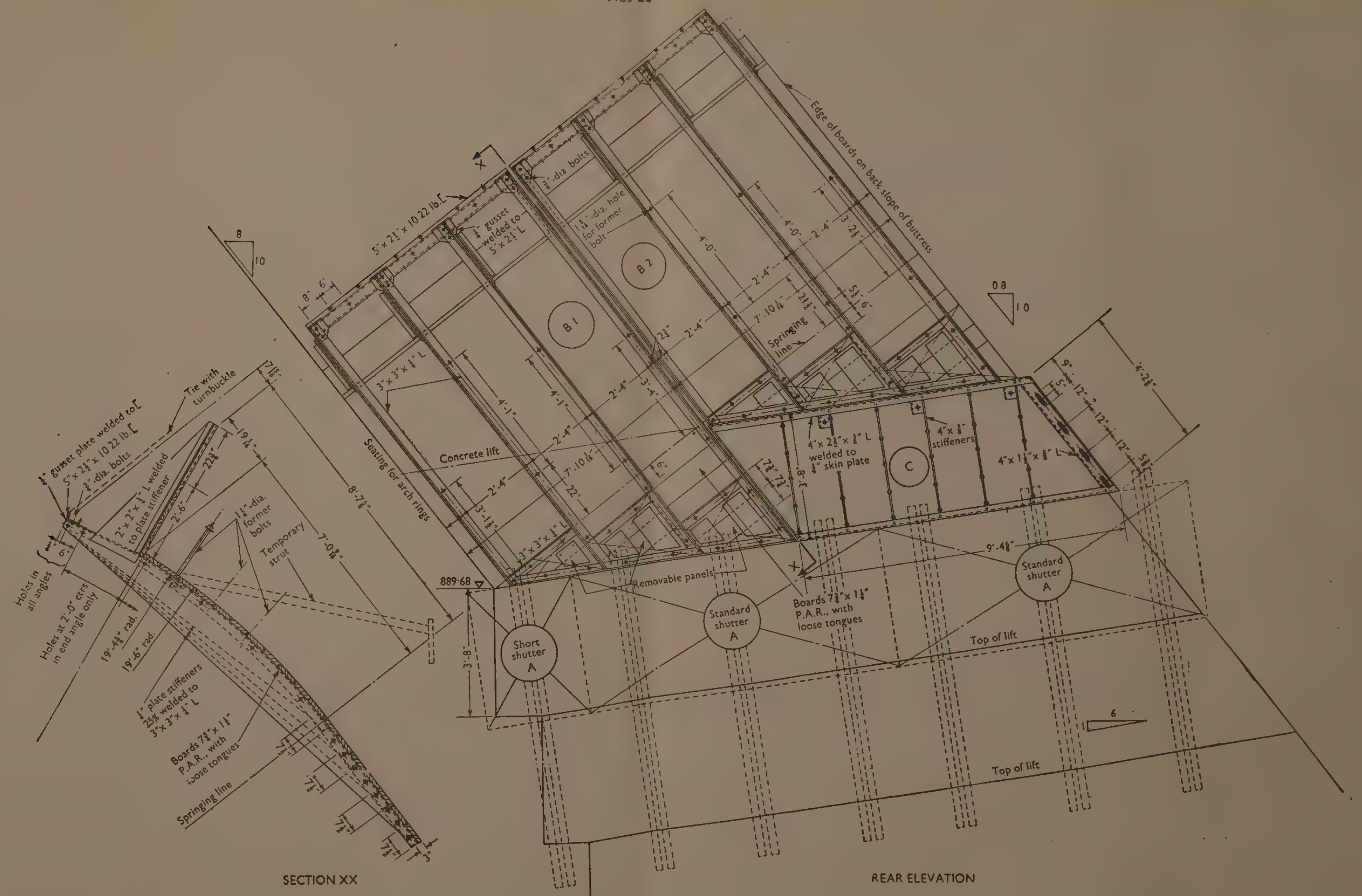
SECTION THROUGH INTAKE

TYPICAL SECTION THROUGH GRAVITY DAM

TYPICAL CROSS-SECTIONS

JAMES STEVENSON

Figs 26



SECTION XX

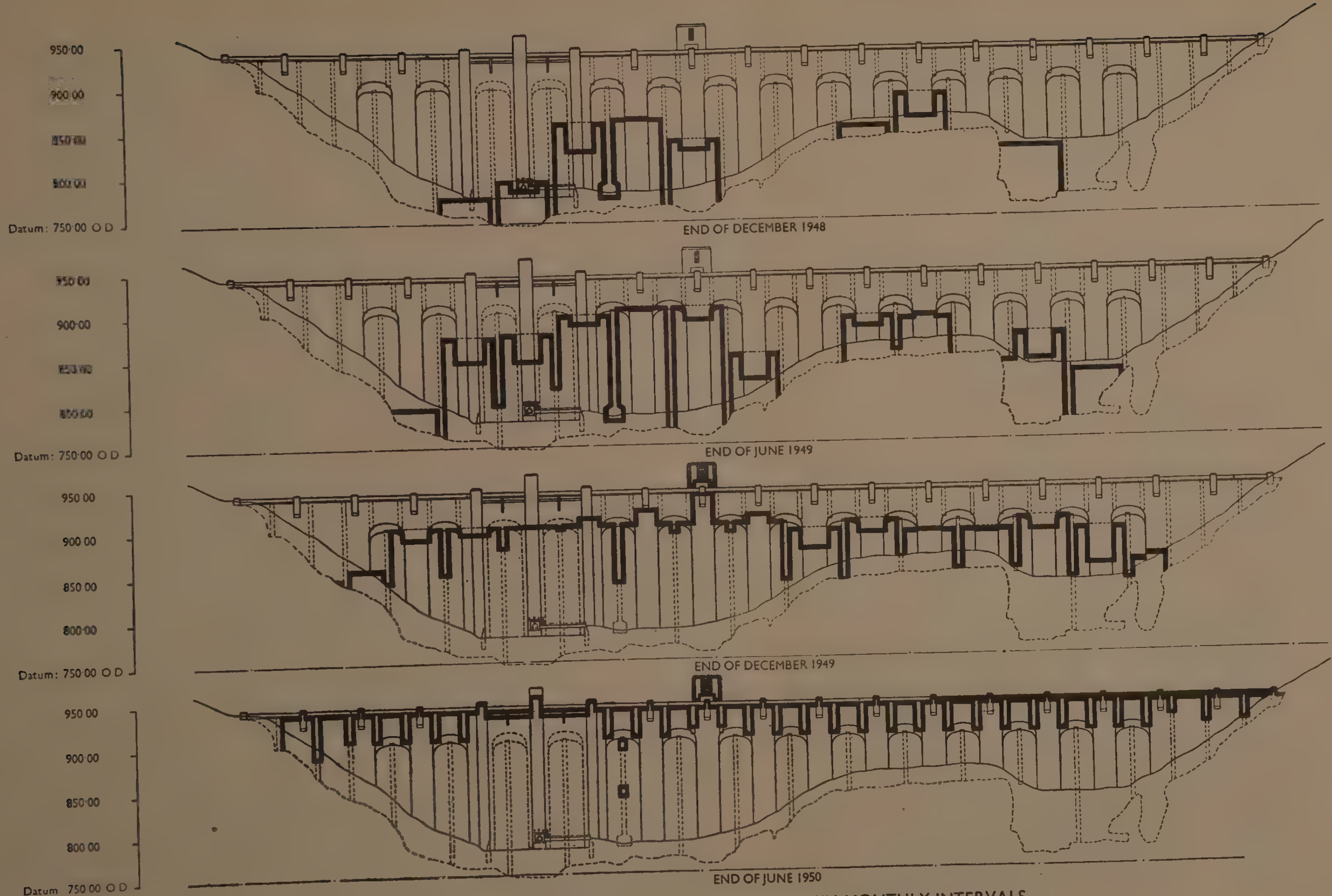
REAR ELEVATION

Scale: 1 inch = 4 feet

ARCH CORBEL SHUTTERS

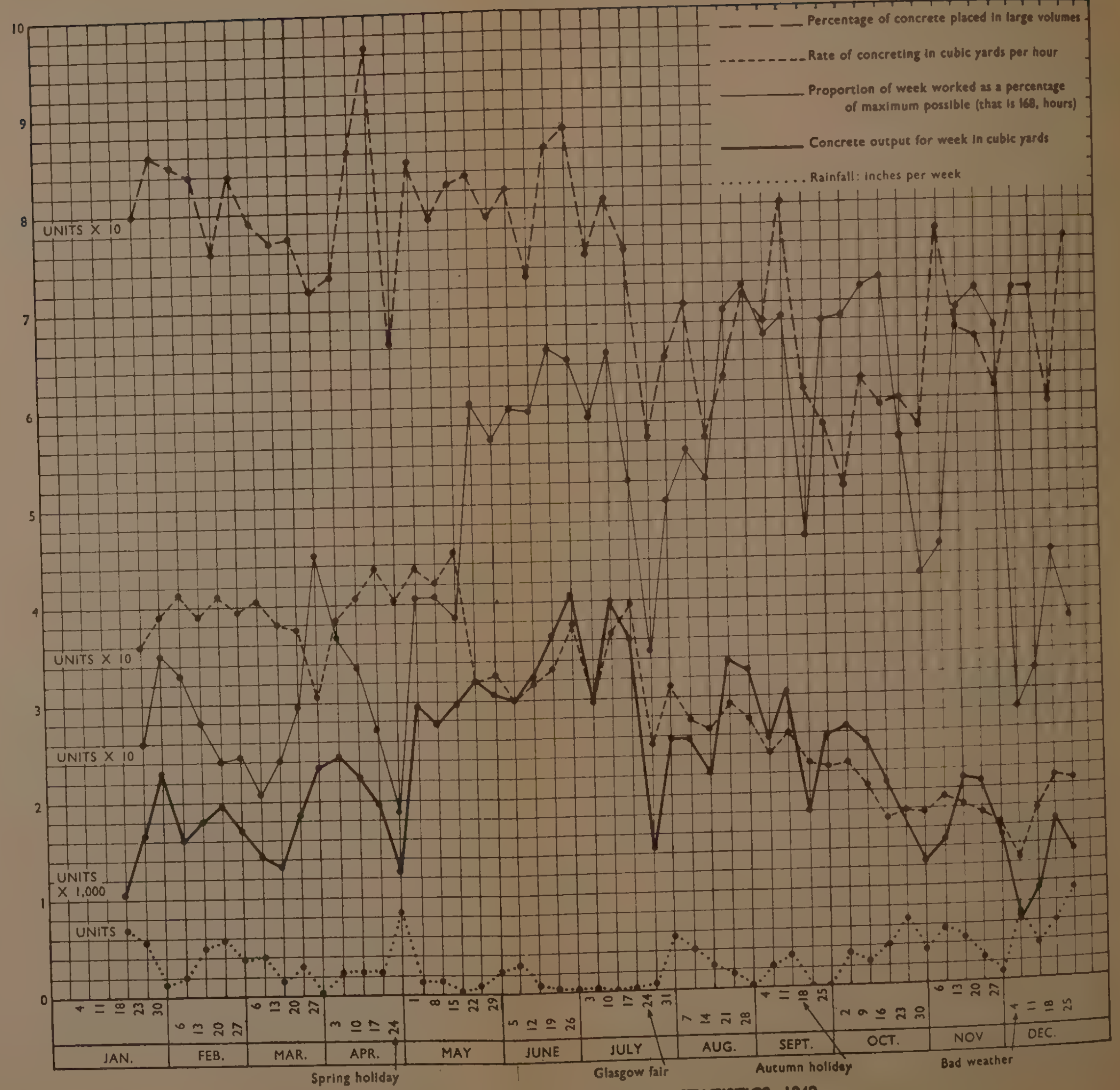
THE CONSTRUCTION OF LOCH SLOY DAM

FIGS 29



CONCRETE PLACING PROGRESS, BY SIX-MONTHLY INTERVALS

FIG. 28



CONCRETE PLACING AND RAINFALL STATISTICS, 1949

The Institution of Civil Engineers Proceedings, Part III, August 1952

& Sons, Limited;

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The Consulting Engineers for the Work were Messrs James Williamson & Partners, to whom the Author is indebted for permission to publish the buttress concrete temperature graph shown on Figs 14.

The Contractors were Balfour, Beatty & Co. Ltd, whose agent at the site was Mr James Hastings. The Author was one of the Contractor's assistant site engineers.

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The Author is also deeply grateful to his employers, Balfour, Beatty & Co. Ltd, for their permission to present this Paper and for their advice and assistance in its preparation (which included the preparation of the drawings) and also for permission to reproduce the various photographs. In particular, for this help, he would like to thank Sir Andrew MacTaggart, Mr D. M. Balfour, and Mr J. D. Gwynn, B.A.I., M.I.C.E.

The Paper is accompanied by fourteen photographs and fourteen sheets of drawings, from which the half-tone page plates, folding Plates 1, 2, 3, and 4, and the Figures in the text have been prepared.

Discussion

Sir Andrew MacTaggart said that the Consulting Engineers had produced a structure which was very pleasing to look at and which at the same time it had been possible to construct in a normal way by methods which were available to all contractors to-day. He thought that it was one of the finest-looking dams in Scotland at the present time.

Remarking upon the weather at Loch Sloy, he said that the maximum rainfall in one year had been more than 160 inches, as compared with about 40 inches in London. That was some measure of the storms which occurred.

He referred to the installation of the cableway by the North of Scotland Hydro-Electric Board before, so far as he knew, the Board had asked any of the contractors who might be interested whether they would like to build the dam. That had been, he suggested, a very great risk to take and a very bold step, but he could give the assurance that it had been a very successful one, so that whoever suggested that innovation had certainly been thinking along the right lines. The other great innovation had been the arrangements for the supply of aggregate from the quarry, which involved one of the longest belts ever built, of a length of about 2½ miles, over very rugged land, with booster motors along the line. It had supplied the aggregate in all kinds of weather, and there had been no hold-up or cause for complaint during the 4 years the contractors had been on the site. That was a very great tribute to the builders and suppliers of the line.

The Paper showed how a young engineer (the Author) had been able to obtain complete command of all that was required to build a structure of the kind in question. The Paper and what the Author had said in introducing it provided evidence that he had a complete grasp of the work and of all that had to be done and should be remembered in future for other works of the same kind.

Perhaps one of the greatest difficulties in the construction had been to get a proper sequence of concreting. The foundation depths ranged from 10 feet to 100 feet in a very short distance, so that the difficulty had been to get a length of trench where it was possible to start concreting and so get a sequence of concreting which would make it possible to carry through a programme in a reasonable period. That had been achieved very well, but it had taken a great deal of planning. On a job of the kind in question it was essential to work out beforehand what the programme was going to be. They had been working on the concreting night and day, and every shift had to have somewhere ready to put the concrete.

Having regard to the small number of carpenters available during the period when the job was being carried out, it had been a great feat to get the shuttering erected by mobile cranes on top of the dam with the help of men who had had little experience of that work. He used to think, when he saw the men at work in all kinds of weather, that even to hold on to the structure was difficult enough without having to do any work. When he said that they had had an average of twelve real carpenters on the job, when they ought to have had fifty or sixty, it might be imagined what a task those men performed, with the few skilled carpenters who had been available, doing work to which they were not accustomed and doing it in such an excellent manner.

Mr William Young observed that points to which more extensive consideration might well be devoted were the planning of the temporary works and the highly efficient and economical method of removing the top surface of the concrete in preparation for the succeeding lift. Possibly the Author could say a little more about that; it was of particular interest, since it had proved to be so satisfactory.

The Paper had included an analysis of the concrete temperature investigations, which unfortunately was not complete, because there was no record of the air temperature during the investigations.* A lot had been written on that subject, and Mr Young had no wish to become involved in a direct comparison with other dams, because there were so many factors which had to be taken into consideration before a true comparison could be made. He would like to confine himself to what he considered to be a significant feature, which was that for the atmospheric conditions in which the concrete had been deposited (an air temperature of about 50° F.), the maximum recorded concrete temperature was just above 80° F. So far as he could see, that was in general lower than previous results in mass dams, and, of course, was not unexpected when one paused to consider the surface area of a buttress dam in relation to its volume. It was of significance, however, because with the lower temperature generated it was logical to assume that the contraction slots could be spaced a little wider than in the case of a solid gravity dam.

That in itself raised two points of interest. There was a popular theory that a buttress dam required not only an appreciably greater amount of shuttering, but also shuttering of a much more complex nature. On the first point, if the Sloy method of forming contraction slots were adopted for each type of dam it would be found that there was much more temporary shuttering in the contraction slots of the gravity dam and that that would off-set the extra finished shuttering required for cladding the greater exterior surface of the buttress dam. The total area of shuttering (temporary and permanent) was therefore about the same in each case.

On the second point, as would have been gathered from the Paper, the complexity of the shuttering had introduced very little increase in the difficulties of construction. The elliptical shutters between the buttresses had, of course, been prefabricated and, once they had been set in position, the raising of them had become a matter of routine which was little more complex than in the case of straight shuttering. It was true that if the arches had been carried out by centering and in-situ methods they would have introduced a comparative difficulty, but the ingenious method which had been adopted of pre-casting and erecting those arches had reduced the construction to one of comparative simplicity.

Before leaving the question of the mass *versus* the buttress dam, it might be mentioned that the question of heat generation in a mass dam had sometimes been tackled by the use of low-heat coarse-ground cement. To-day, however, not only was there difficulty in persuading the manufacturers to supply that cement, but also it involved, as was well known, a substantial increase in price. At the moment the increase was of the order of 21*s*. per ton and when that was applied to the volume of concrete involved, the increased cost would practically pay for the total cost of shuttering a buttress dam. There was no question of calling a buttress dam a substitute for a gravity dam. It was not so, but the arguments which he had put forward were of interest on a site which might be adapted to either type.

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* Records of maximum and minimum air temperatures have been incorporated in *Figs 14* since Mr Young made this comment.—Sec. I.C.E.

One point in the Paper itself had puzzled Mr Young. It would be found that the Contractors had elected to raise their lifts at 3-foot-8-inch intervals, while the Engineers had specified that they could go up to 5 feet. It occurred to him that that had involved a considerable excess of scabbling of surface areas and preparation of surfaces, the cost of which was included in the concreting cost, and it seemed to require some explanation.

He concluded with a remark on the weather. He agreed that the recorded rainfall was in itself graphic enough, but to appreciate the conditions fully it must be realized that the rain was invariably coming in horizontally with gale-force winds. He would like to record his own tribute to the determination of the men who worked continuously and so well under those arduous conditions.

Mr J. D. Gwynn observed that in any dam the importance of the shuttering varied as the job progressed. At first the speed of pouring the concrete determined the rate of progress, but a stage was soon reached where what mattered most was the rate at which it was possible to set up a fresh lift of shuttering. That remark applied with unusual force in the case of the Loch Sloy Dam, because it consisted of a buttress section surmounted by a small gravity section. The result was that just as the cross-section was getting thin and the shuttering had to be set up with increased frequency, it had been necessary to introduce a number of changes almost at once. The elliptical shutter was no longer used, the arch rings had to be put in, pilasters had to be added, a vertical curve in the back of the dam had to be introduced, and, to complicate the matter further, the position of the joints had to be altered and their number increased. Even if an unlimited number of carpenters had been available, there was a very dangerous potential bottleneck there, and the pre-casting of various units had been part of the attack on the shuttering problem.

The shuttering itself had been the object of a considerable amount of thought, and it had been found that the importance of having shuttering which was simple and reliable, and which could be quickly and flexibly altered and adjusted to all the different dimensions required when a dam was going up with sloping faces on both sides, far outweighed any possible saving obtainable by adopting a cheap shutter. If a shutter of the full maximum height had been used, as Mr Young had suggested, the unit would have been clumsier. For example, it would have been necessary to cope with correspondingly longer soldiers of heavier section to get the same control over the shutter. The length of a soldier to control a shutter of the type chosen had to be approximately 2½ times the height of the shutter. Enough had been said to show that the weather conditions did not encourage having objects rising high above the concrete surface. On consideration, Mr Young might agree that there were advantages, in spite of the extra scabbling, in having something which did not take too large a bite at a time.

On 13 April, 1948, when those shutters were being assembled, a Paper[1] had been given to the Works Construction Division by Mr Cyril Parry. It was a very interesting Paper, but Mr Parry's conclusions on the subject of heavy forms such as those in question were in striking contrast to the arrangements which had been used at Loch Sloy Dam. Mr Parry had stated: "Heavy forms of $\frac{3}{16}$-inch plate, or thicker, are best assembled with rivets, countersunk, chipped or ground off on the face side. The riveting should be carried out with special care in order to avoid denting of the plates around the rivets. The Author has often been persuaded to allow manufacturers to produce special panels by welding the channels or angles to the plate, and he has always found such panels unsatisfactory; this may be due to lack of skill of the welder. The resulting panels are usually inaccurate, owing to warping and twisting of the units. It is advisable that steel form plates should have their edges machined and not sheared, and that they should be made to a tolerance of minus $\frac{1}{64}$ inch and plus 0. When this degree of accuracy is asked for, welding appears to fall short of requirements." With regard to that, Mr Gwynn would point out that the shutters in the Loch Sloy Dam—not only the flat ones, but also the horizontal and vertical curves—were all made of welded plate construction. It would be interesting to know, in view of what Mr Parry had written, whether the Author had found them satisfactory in practice or otherwise. With regard to tolerances, it was possible to do a welded job to a tolerance of $\frac{1}{64}$ inch, but it was generally more economical and profitable to arrange the design so that the soldiers and individual shutters could be adjusted to get the required degree of accuracy in the set-up, rather than to spend a great deal of money in getting very accurate dimensions where they were not, perhaps, necessary. That was the basis on which the shutters had been designed, with the exception of a few which had been of mixed timber and steel construction, owing to the fact that they had been used only a few times.

Mr Alan P. Lambert said that he had had the good fortune to visit the work twice while it was being carried out. He had experienced a sense of real exhilaration when he saw the dam at Loch Sloy. He thought that here at last was a British dam which did not look exactly like the cross-section of a piece of cheese. He thought that it was impossible for anybody to stand below the Loch Sloy Dam and look up at the buttresses, crowned by the arches, without feeling that it was an inspiring design in which everybody should take great pleasure.

The other impression which he had gained was one of profound respect for the Contractors' site staff, who had carried out the work in the most appalling conditions and had been determined to do the job, if possible, ahead of schedule. It was perhaps not always fully realized how much the successful conclusion of big projects in outlandish places really depended on the determination and "guts" of the site staff. They had to deal with most of the troubles and had to coax the labour along, but never allowed themselves to be depressed. The Author was to be congratulated not only on an admirable Paper but also as a member of a team who were entitled to look back on their achievement with pride and satisfaction.

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[1] Cyril Parry, "Shuttering." J. Instn Civ. Engrs, vol. 30, p. 303 (Oct. 1948). See p. 316.

There were a few points in the Paper on which Mr Lambert would like to ask some questions. The maximum size of aggregate had been 2½ inches. Both economically and technically there were considerable advantages in using a much larger size of aggregate, possibly 6 inches or even 9 inches. If it were only necessary to crush half the stone down to 6 or 9 inches there would be economies in crushing costs, and also the total specific surface to be covered by cement would be reduced; that might lead to an economy in the quantity of cement used, which would bring in its train the technical advantage of reducing the temperature rise in the mass, with the corollary of less risk of any serious cracking developing as the mass cooled. Mr Lambert was not asking why the Author had not used a bigger aggregate, because as a good contractor the Author had naturally used the aggregate which he had been told to use; but it would be interesting to know whether, if he had been asked to use an aggregate of 6-inch maximum size, he would consider that it would introduce any serious problems in the process of mixing, distributing, and compacting the concrete. Certain types of mixer could not handle material of that size, but the technique adopted should be adapted to make use of the maximum size of aggregate suitable for the structure which was being built, and that size should not be controlled by the problems of the actual mixing and placing of the concrete.

Table 1 gave a great deal of information about the concrete, but there was one result there which called for an explanation, namely, the rather poor performance put up by the second mix. For the greater part of the dam a mix with 3¼ cwt of cement to a certain proportion of sand and stone had been used, but it was mentioned that for certain situations a higher-strength concrete had been required, and the second mix had been introduced, where the cement had been increased from 3¼ to 4 cwt, and the water/cement ratio had been brought back from 0·70 to 0·60. Those adjustments should have resulted in a concrete with a strength of more than 3,000 lb. per square inch, but in fact the strength of that mix had been about 200 lb. per square inch less than that of the weaker concrete. That did not relate to the result of a sporadic cube test, but to the average strength covering 3,000 cubic yards of concrete. Mr Lambert had checked the density figures, thinking that for some reason the mix in question might have been difficult to compact, but, taking the specific gravity of the cement as 3·1, which seemed to be a fair figure, and using the other data given in the Paper, it would be found that the maximum possible lensity of the second mix, assuming the complete elimination of all voids, was only 156 lb. per cubic foot, and under test a density of 155½ was btained. That was very satisfactory and showed that the mix could be ully compacted, but it still left unexplained the mystery of the low trength.

Bulk cement was one of Mr Lambert's hobby-horses, and he was certain hat the Contractors in the present instance had been fully aware of the reat advantages of using bulk cement, which lay not only in the saving f bags but in the very considerable saving of handling costs at the ite. No doubt the Contractors had wanted to use bulk cement, and it would be interesting to know what had led them to abandon it and ccept the cement in bags. If it had been a question of transport diffiulties, those had been overcome on another dam. If it were a question f the Cement Marketing Company not wanting to supply in bulk, they ught to know about it.

If the 5-foot shutter had been used it would have meant a reduction f about 20 per cent in the number of lifts, with the advantages to which n earlier speaker had referred. At the Loch Sloy Dam, 2-ton cranes had een provided to lift the shutters, so that it was not a question of the weight f the shutters which had caused the smaller lift to be adopted.

The belt conveyor which had been referred to by Sir Andrew MacTaggart ad not been the choice of the Contractors, but it was encouraging to hear hat it had been an extremely reliable piece of equipment. With twelve onveyors in line, if one broke down the whole train came to a stop, and it was interesting to hear that that had not happened very often. At first ight it might have been thought that, with a road to the quarry and to he site of the dam, the more obvious method of using big lorries for ransporting the material might have been preferable.

The cableways were very interesting, but unfortunately Mr Lambert's isit had been on one of the rare occasions when one of them had broken own and was under repair. He was not certain to what extent the use of wo caused interference of one with the other. The upstream one could ot return to the loading point if the downstream one was upstream of the oading point. He did not know whether that would be a complication in ractice, or whether everything worked smoothly when some experience ad been gained. It was interesting to see that it had been found at Loch loy to be the right system to use a transfer skip running from the mixer nd discharging into permanent cableway buckets, whilst at another dam, sing the same cableways, the cableway skips had been changed and run ackwards and forwards to the mixer.

In conclusion, he would like to pay his tribute to the extremely satisactory way in which pre-cast units had been worked into the dam. It owed a very intelligent appreciation and a happy state of co-operation hen the designer was prepared to consider proposals of that sort. In articular, the use of the arch ribs was a brilliant idea. The fixing of the soffit falsework for the arches might not have been too bad, but getting it out afterwards would have been a more serious problem.

Mr J. C. Waddington wished to know if the Author had had any serious trouble with the cableways in high wind. Mr Waddington's experience of cableways had been that in fairly high wind there was a considerable amount of swing of the concrete-bucket, especially when concreting at a considerable depth below the level of the cableway. There was also a fair amount of surge in the cableway when the hopper of the bucket was released and the concrete flowed out. It would be interesting to know whether any troubles of that kind had been experienced. The Paper described how some of the difficulties had been overcome, but it was Mr Waddington's experience that the difficulties could be rather serious in high wind. From what was said on p. 193 of the Paper he noticed that although the specification permitted the use of plums up to a maximum size of 4 tons, they had not been used in the work in question. He fully agreed with that procedure, and he was very pleased to see that the plum bogey had been laid again. The ability to use plums up to 4 or 5 tons in weight had attractions for contractors and looked very nice on paper, but in practice one was faced with the difficulty of quarrying them, and the need for heavy plant to handle them and transport them to the site and for heavy derricks to place them on arrival at the dam site. That often meant that each bay had to be served with at least two derricks, or a derrick and a cableway—one of the derricks for placing the plums and the other or the cableway for placing the concrete. His experience was that even with a fairly broad specification, allowing plums up to 5 tons in weight, with a granite quarry fairly close to the site of a particular dam, only 31 per cent of the volume of the concrete in the dam had been taken up by displacers or plums.

The adoption of aggregate of larger size, which had been recommended by Mr Lambert, had a great deal to be said for it. It had been used to a large extent in America, where the employment of what were referred to as "cobbles" up to 6 or 8 inches in size was practised. The use of "cobbles" would reduce the latent heat of the concrete and produce a smaller surface on which the cement mortar had to adhere.

The Contractors and the Engineers were to be congratulated on the way in which they had overcome the difficulties of concreting and shuttering the top of the arches. With the great scarcity of carpenters, there had undoubtedly been serious difficulties facing the Contractors, and there was much to be said for the co-ordination shown between the Engineers and Contractors in scrapping the original design and substituting a feature which appeared to have great advantages to both parties. Mr Waddington thought that the use of pre-cast work might well be adopted on a large scale for mass-concrete dams of varying descriptions, and particularly for parapets, parapet walls and other items high up on the dams.

What was the Author's view on a comparison between the buttress-type dam which had been built and a solid dam? Did the Author consider that the economies in concrete had been worth while in the buttress dam as compared with a solid mass-concrete dam? The mass-concrete dam had been referred to by Mr Lambert as looking like a cross-section of a piece of cheese, but something which could be compared to a cross-section of a piece of cheese obviously did not require the same amount of shuttering as a buttress-type dam.

Finally, would the Author be willing to provide some costs of excavation (soft and rock), shuttering, concreting, and pre-cast work? For some reason or other there seemed to be very great reticence about giving costs in Great Britain. It was difficulty to see why that should be so, because the production of costs was more likely to lead to a wider realization of what such jobs really did cost, and would tend to keep the economy both of engineers and of contractors in better order.

Mr Peter Campbell, referring to the contraction slots, recalled that on the last occasion on which he had visited the Loch Sloy Dam there had been two Spartans standing in one of those contraction slots treating it in some way with a huge blowlamp and bitumen. He called them "Spartans" because they were working in what was perhaps the most exposed part of the whole dam site and the horizontal rain which had been referred to was driving down the valley and, although it was the month of May, there was sleet as well as rain in the wind. Would the Author explain exactly what treatment had been given to the contraction slots?

Mr W. J. H. Rennie observed that he had been interested in the contract in the early stages, when his firm had been considering tendering. At that time it had not been decided to let out the contracts in sections, such as the road, the camp, the quarrying, crushing, transporting the aggregates, and so on. It had then been decided to provide cableways and, since that had taken some of the interest out of the work from his point of view, and added to the labour difficulties, his firm did not in the end submit a tender.

Mr Rennie did not know what the cost of the cableways had been. Their function had been confined to the concreting, including the setting of the pre-cast arches. He estimated that the cost of the cableways—depending upon how much one depreciated the plant—might amount to as much as 5*s* per cubic yard. He was glad that Mr Waddington had raised the question of costs, because in a construction Paper one might expect something to be said about the cost of the work.

Taking the data given by the Author in Fig. 28, Plate 3, and applying the figures to the whole of the concreting programme, the maximum average output over the recorded 12 months—the main concreting period—appeared to be about 2,200 cubic yards per week, or about 26 cubic yards per concreting hour. The overall average for the whole concreting period appeared to be 1,450 cubic yards per week, and that meant that the two

10-ton cableways were provided to deal with an average of approximately 17 cubic yards per concreting hour, which seemed to be a very low figure.

Mr Rennie's own experience with cableways had been rather a mixed one, and he would like to know whether, as a result of their experience and if left to their own ingenuity, the Contractors would have provided cableways. Sir Andrew MacTaggart had spoken very highly of those which had been provided,

Almost every speaker had had something to say about the weather. Personally, having spent some years in other parts of the world where a rainfall of 100 inches per month was not uncommon, he found 163 inches in a year, though quite considerable, not alarming. During one job with which he had been concerned an intensity of 20 inches in 24 hours had been recorded.

The average working time in Fig. 28, Plate 3, was given as a percentage of a maximum possible of 168 hours. That was perhaps a little misleading. Mr Rennie did not know the number of shifts that had been worked, but he assumed not more than twelve, and the working shift would probably be 10 hours. On that basis, which would give 120 hours, the average working time was increased from about the 50 per cent on 168 hours to about 66⅔ on 120 hours. That showed a loss of about one-third (say 33 per cent) of the working time. To those who knew the Highlands that might not appear excessive, but the Author had drawn attention to the appalling weather. Normally, Mr Rennie would expect that the inclement weather might account for something of the order of 10 per cent from rainfall, and from gales and frost probably another 10 per cent, so that on his figures there was still a difference of about 13 per cent presumably attributable to abnormal weather.

The General Conditions of Contract agreed between the Institution of Civil Engineers, the Association of Consulting Engineers, and the Federation of Civil Engineering Contractors made abnormal weather conditions a contractual risk, which in Mr Rennie's view was wrong. Since that contract, however, had been let in the days of the Standard Form of Contract, first edition, by which the weather was not excluded from the physical conditions, it would be interesting to know how much the Contractors would allow for abnormal weather conditions if the contract were being carried out now.

Mr J. M. Fisher referred to one of the slides, shown by the Author, illustrating the arrangements made for the supply of aggregates to the Loch Sloy Dam, in which a stockpile had been shown on the top of a collecting tunnel. Mr Fisher understood from various technical Papers that that method of tipping aggregates on to a pile was likely to lead to very serious segregation of the particular grades of aggregate, and he asked whether in fact very big variations in the workability had been noticed as the stock piles were run down.

Table 1, in which the details of the coarse aggregate were given, inditated that the $2\frac{1}{2}/1\frac{1}{2}$-inch size was somewhat less than the $1\frac{1}{2}/\frac{3}{4}$-inch size, and somewhat more than the $\frac{3}{4}/\frac{3}{16}$-inch. If the recommended grading curves given for concrete with different maximum sizes of aggregate were examined, it would be noticed that the maximum size which was used, as compared with the other sizes, whatever they might be, was always something of the order of 2/1/1. Had extensive trials been made with the particular mixes concerned to ensure that the best results were being obtained with the overall mix proportions used ?

Mr Fisher had not been able to make a very careful analysis of the results of the tests which had been made, and the Author's amendment to the minimum permissible strengths, given at the beginning of the discussion, put a rather different complexion on the variations which arose n the course of the work. The ratio of the actual average strength to the minimum permissible strength was about 1 : 3·3 for the grade of concrete forming the bulk of the work ; that indicated that quite a high degree of control had been maintained despite the weather. The Contractors were to be congratulated on attaining what appeared to be remarkably good igures during the course of the job.

Mr N. G. Eggleton, referring to Mr Gwynn's contribution, asked the Author whether he would use steel shutters of the same welded construction on a smaller structure where a first-class finish was essential. Mr Eggleton thought that Mr Parry's remarks on the dangers of welding steel shutters should be taken as referring to their manufacture for use on buildings where a first-class finish was essential.

*** **Mr R. E. J. Worth** observed that the method employed of cleaning-off the surface of freshly poured concrete by means of a high-pressure air and water jet was of interest but had some disadvantages upon which the Author's opinion would be helpful.

In Mr Worth's experience the timing of that operation was very critical n that a premature application of the jet could cause considerable damage o the pour, whereas if it was left too long the jet would not have the desired effect ; the effective period between those two extremes seemed o be comparatively short—between 1 and 2 hours. Good supervision of he operation was therefore essential. If continuous shift working was in operation on the job that requirement was not difficult to arrange, but if he job was on single shift and most of the concrete pours were finished at he end of the day's work, the jetting operation required men to be turned-out during the night ; that in turn could easily result in the occurrence of ither of the two extremes mentioned above unless the cost of fully adequate dditional supervision was accepted, especially under adverse weather conditions where the tendency of the jetting-gang was, quite naturally, to et the job done as early as possible and then return to shelter.

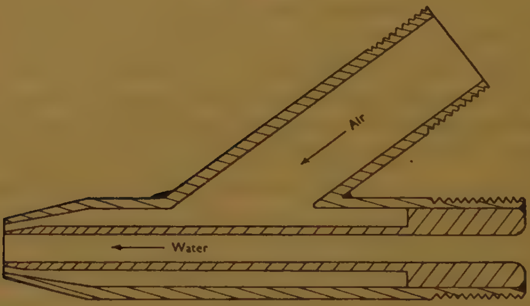
Could the Author give any indication of the timing of the operation and whether temperatures affected the timing to any extent? Also had both air and water been supplied under pressure, and if so at what pressures, or had only the air been under pressure?

*** This contribution was submitted in writing.—Sec. I.C.E.

The Author, in reply, endorsed Sir Andrew MacTaggart's high opinion of the aggregate conveyor. No hold-up had been caused by inability to deliver aggregate to the batching plant. The contractors' own conveyors from the aggregate stockpile to the batching plant had occasionally broken down, but on those occasions the Monotower crane had been used to load coarse aggregate into the storage hoppers. The Board's conveyors had also, on rare occasions, broken down, and there had been an occasional stoppage at the quarry, but there had always been ample supplies available in the stockpiles to enable concreting to continue.

Mr Young had been impressed by the efficiency of the operation in which laitance had been removed from the top of the lifts. It had indeed been very successful, and there was no doubt that it had produced a very great saving, since very little subsequent picking or scabbling of the surface had ever been required. The washing had been done by means of a very powerful jet of water and compressed air, applied simultaneously to the concrete from concentric nozzles (*Fig. 30*) held a foot or so above the top of the lift a few hours after concreting. The nozzles had been fabricated on site, and control valves provided at hand level on both water and air leads.

Fig. 30



Scale: ½ full size

Concentric Nozzle used for Washing Laitance from Top of Concrete

The concrete temperatures shown in *Figs 14* had been obtained from ordinary maximum/minimum-type thermometers placed in the centre of the buttress lifts through 4-inch-diameter steel pipes which had been cast into the buttress concrete from the edge of the buttress to the centre of the lift.

The question of the optimum height of lift was extremely interesting

As Mr Gwynn had said, the decision to adopt a 3-foot-8-inch lift had been taken because the resulting shutter was a convenient size and fitted in very well with the requirements at the higher levels. That decision had, however, been taken in the very early stages of the job and, with all due respect to Mr Gwynn, the Author thought that there might have been savings had the full permissible depth of lift been adopted. No doubt the shutters would have been more difficult to handle, but, as both Mr Young and Mr Lambert had pointed out, there would have been savings elsewhere—if not in the shutter itself, at least in a reduction in the number of lifts and in the costs of cleaning and washing off (though that had been carried out easily), and also a reduction in overall shutter standing-time. It might be of interest to mention the quantities of concrete involved in lifts. At the lower levels that was as much as 300 cubic yards in a few sections, but at the higher levels some lifts contained only about 45 cubic yards of concrete.

The welded shutters stood up to the job very well and were more rigid than if they had been riveted. Very little repair work had been carried out on the shutters, and what there was had been almost entirely confined to dealing with accidental damage.

Mr Eggleton had asked about the finish obtained, and if, where precision work was required, the welded shutter would produce the required finish. The Author could only say that on the Loch Sloy Dam an accurate and first-class finish had been obtained with the shutters that had been used.

The Author appreciated the tributes paid by Mr Lambert and others to the appearance of the structure, which he thought had contributed a great deal to the interest which everyone had taken in the job, and, no doubt, to the progress of the work as well.

The maximum size of aggregate had been limited to 2½ inches by the specification, and, in any case, that had been the maximum size produced at the quarry. With aggregate of larger size there would have been certain economical and technical advantages, as Mr Lambert had pointed out. The mixes could have been denser and of higher workability, and less cement would have been required, resulting in reduced shrinkage. A completely redesigned batching plant, however, would have been necessary; an increase in the number of storage hoppers would have been required; and batching-up would have been more complex with the increased number of gradings. As it was, with 2½-inch maximum grading, constant heavy maintenance to the mixers was required, particularly to the blades and rollers. It had been, he thought a mistake to mount the mixers on the frame of the batching plant and not on separate foundations, because considerable vibration had been transmitted throughout the structure. That question of the maximum size of aggregate rather indicated that economies might well be effected by leaving the design of mixes entirely to the contractor, and merely specifying minimum requirements of the finished concrete, such as strength, density, impermeability, etc.

In Table 1 the 4/2½ cube test, particularly at 28 days, had given lower results than should be expected. There was a logical explanation for that, as Mr Lambert had thought there would be. Almost all Class 4 concrete had been placed in the early stages of the job around the 13-foot-6-inch-diameter steel pipe at the intake. That had not been spread over a long period, and so relatively few test cubes had been taken, and of those only very few had been crushed at 28 days ; one or two had been slightly under strength, and the others just slightly over strength, giving the average figure shown in Table 1. At that stage, the test cubes had not been cured thermostatically, so that faulty curing might well have contributed to the low strengths obtained.

The use of bulk cement had, of course, been investigated. It had not been adopted, because the suppliers in the area at the time had not had the facilities available to handle it, and could not, therefore, guarantee continuous deliveries of the large quantities involved.

During the early stages of the work there had been little, if any, interference between one cableway and another, but later on detailed planning had been necessary in order to avoid interference, especially after the transfer track had been shortened. In the last 2 or 3 months of the work it had been impossible to avoid interference, and during that period only one cableway could be operated really efficiently at one time. Mr Lambert had also mentioned the possibility of running the cableway skips right down to the batching plant. That had, in fact, been tried and timed in comparison with the method which had eventually been used ; the transfer had been found to be speedier.

Mr Waddington had asked if there had been trouble with the cableways in high wind. Very occasionally it had been necessary to stop work because of the wind, but the cableways were usually amongst the last items of plant to stop work. The cableway drivers had handled the cableways very well and had done much to minimize surge when the load was being hoisted and after the concrete had been deposited, during periods of high wind. The particular type of drop-bottom skip used also greatly reduced surge after concrete had been deposited. The fact that there had been less surge than one would normally have expected had a somewhat peculiar and, unfortunately, disastrous effect on the button-rope flexible ends. These had had to be replaced from time to time, because with so little surge there had been very little rocking action or movement in the pulleys and the unit wires were thus kept in compression at the pulley face without relief or stress, and fracture finally occurred at those wires at the pulley face. The cableway manufacturers had now rectified that by a slight modification in design.

Mr Waddington had questioned the relative economies of the buttress type as against the mass-concrete gravity type of dam. The Author could not say much about relative costs, but he pointed out that had the dam been of the solid mass-concrete type, shutter lifting would have been much speedier and far higher rates of concreting could have been achieved. As it was, on the gravity-dam section at the hillsides progress was normally about twice as rapid as on the more awkward buttress sections.

The Author would not like to say much about costs, but he mentioned that the total value of the contract was £1,351,667, of which £144,800 had been carried out by daywork. The daywork included the construction and dismantling costs of the cableways. Various quantities were given in the Paper for excavation, concrete, and shuttering, and perhaps Mr Waddington would like to work back from those figures!

Mr Campbell had asked about the treatment of the contraction slots. The sides of the contraction slots had been scabbled with a small rotary-type scarifying tool driven by compressed air, and the three key checks at the lower levels and the two at the higher levels had been particularly carefully scabbled to remove laitance and yet leave a quite smooth surface. A bituminous sheeting had then been applied hot to the key checks, using a concrete priming solution.

Mr Rennie had had a rather mixed experience with cableways, and wondered if the Contractors would have provided cableways had they been left to their own ingenuity. When preparing the tender for the job, the Contractors had seriously considered the alternative of providing a gantry and derricks. On paper, that looked attractive when compared with the estimated cost of operating the cableways. However, having once experienced the cableways in operation under the site conditions, the Contractors were extremely thankful they had not adopted the gantry method. At times the cableways had been hard put to it to cope with the amount of concrete which could be produced, and, provided the shuttering could be kept ahead of concreting, the amount of concrete which could be placed had, to some extent, and for short periods, been limited by the handling capacity of the cableways. On the whole, however, that had not been true, and the cableways had been quite equal to the job. They had been invaluable when placing the pre-cast work, because about 99 per cent of the weight of the units could be taken by the cableways so that the units could be pushed by hand into position. Generally, concreting had proceeded very smoothly, as a routine business. The problem had not been one of the actual production and placing of concrete, but rather of getting the lifts prepared for concrete; that was to say, getting the foundations finally trimmed and passed, shutter lifting, and setting. In neither case would methods alternative to the cableways have produced speedier work under the site conditions. It had been found that the cableways could continue placing concrete when derrick work and shutter lifting had stopped owing to the weather. The Contractors had had considerable experience of cableways manufactured before the war, and the improvement in the design and operation of the new cableways had been very marked. After their experience at Sloy, the Contractors, f faced with similar site conditions, would prefer cableways to the gantry and derrick method. Once erected, the cableways were very economical to operate.

Mr Rennie had analysed the data on Fig. 28, Plate 1, which applied to the year 1949. He had spread his findings over the total period of concreting and had deduced the average concreting rate of approximately 17 cubic yards per concreting-hour, which was very low, and much lower than the actual rate which had been achieved per concreting-hour. Perhaps Mr Rennie, when making his calculations, had taken a year in which concreting had been carried out on double shifts, and spread it over the remaining concreting period in which concreting had not always been double-shifted. Up to the time when the transfer track had been shortened, the following average concreting rates had been achieved :—

Single-shift : 38 cubic yards per concreting-hour
Double-shift : 31 cubic yards per concreting-hour

but after the shortening of the concrete-transfer track the average concreting rate dropped to approximately 20 cubic yards per concreting-hour.

Mr Rennie had thought that the rainfall of 163 inches a year was not particularly alarming ; one job with which he had been concerned had had an intensity of 20 inches in 24 hours. The Author pointed out that it was not the maximum daily intensity of rainfall that was so alarming at Sloy, but the fact that it rained almost continuously. During the whole period of concreting only 3 completely dry weeks were on record.

A general indication of the reasonable allowance which could be made for abnormal weather conditions was provided by the fact that the effect of rain and wind on the shutter-lifting gangs was such that it was rarely possible to do serious concreting for more than 4 days in a week.

Mr Fisher had asked about segregation in the stockpiles. Gradings did vary slightly from time to time, but the effect was not sufficiently marked to be troublesome. Since there were no division walls between the various stockpiles, care had, of course, been necessary to ensure that some material was not drawn from the wrong stockpile. Regarding the details of the coarse aggregate gradings, very extensive trials had been carried out before the particular gradings used had been adopted, and $2\frac{1}{2}/1\frac{1}{2}$-inch batch proportion had been reduced to the figure shown in Table 1 to eliminate segregation.

Mr Worth had pointed out that the timing of the operation of cleaning-off the surface of freshly poured concrete by means of a high-pressure air-and-water jet was very critical, and the effective period in which it could be done was comparatively short. At Sloy it had, perhaps, been so successful because it had been treated as an extremely important operation, warranting the full-time attention of a sub-foreman and at least two gangs of men. It had been essential to double-shift that work, so that there was always a gang available for washing at precisely the right time. The Author would not like to lay down exactly at what times the operation had been carried out, because that was greatly affected by the atmospheric emperature, but in practice the ideal time could easily be spotted when he aggregate was sufficiently firmly held yet laitance would wash off. Air had been applied at the pressure from the air main (100 lb. per square nch) and water as delivered from the water main, which, at higher levels, was not under much pressure.

Correspondence on the foregoing Paper is now closed and no further contributions, other than those already received at the Institution, may be accepted.—Sec. I.C.E.

STRUCTURAL AND BUILDING ENGINEERING DIVISION MEETING

5 February, 1952

Professor A. J. S. Pippard, M.B.E., D.Sc., M.I.C.E., Chairman of the Division, in the Chair.

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Structural Paper No. 30

"Some Recent Experience in Composite Pre-cast and In-Situ Concrete Construction, with Particular Reference to Pre-stressing"

by

Felix James Samuely, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

A general outline is given of the principal advantages and disadvantages of pre-cast construction with in-situ concrete, to show that most advantages can be retained and disadvantages avoided by using composite construction.

The problems of adhesion between pre-cast and in-situ concrete and the difficulties of transporting long thin pre-cast units are discussed. Practical solutions are given together with results of recent tests on castellated composite beams.

It is shown that composite pre-stressed beams, with the tension zone pre-stressed, are more economical than other types of pre-stressed beams. Standard pre-stressed soffits have been developed, and calculations for T-beams using these soffits are given, taking into account the variation in Young's Modulus between the in-situ and pre-stressed concrete. Results of a full-load test on such a beam are in close agreement with the calculations.

Applications of composite construction in Great Britain and abroad are illustrated and described, special reference being made to the use of pre-stressed soffits in continuous structures.

INTRODUCTION

For some time past pre-cast concrete has been used frequently to replace constructions which, formerly, were carried out in in-situ concrete. Pre-cast concrete was first developed for slabs, but it is now being developed for beams, columns, and rigid frames (and even space frames)—the intention being to replace in-situ concrete as much as possible. This development, born out of economic necessity, has recently been fostered in Great Britain by these circumstances :—

(1) The shortage of timber shuttering since the war—a shortage that is likely to continue for a considerable time to come.

(2) Social conditions, particularly due to full employment, which have influenced workmen, especially skilled workmen, to seek employment in workshops and factories rather than at the site (with the more difficult working conditions), and have thus caused acute labour problems, particularly in country districts.

(3) The present trend towards standardization.

The advantages of pre-cast concrete can be summarized as follows :—

(*a*) Saving in shuttering which, even when it is readily available, constitutes a heavy expense which is completely unproductive.

(*b*) The possibility of introducing more complicated members than the usual rectangular beam or column. The introduction of different shapes is often of great value both structurally and aesthetically, but with conventional shuttering they cannot be entertained. Two examples are latticed construction and I-beams.

(*c*) Better conditions for the pouring of the concrete, which leads to improved quality and the use of higher permissible stresses.

(*d*) Improved finishes owing to working under factory conditions. Special types of finishes can often be achieved more easily, since several different kinds of concrete (ordinary concrete and coloured casing) can be used more readily in pre-cast units than in in-situ concrete.

(*e*) The possibility of using the labour where it is most available and thus bringing as few workmen as possible to the site.

Against these must be considered the following disadvantages :—

(*a*) Increased transport costs. First the aggregate and/or cements must be taken from the quarry or factory to the manufacturer of pre-cast concrete, and then the finished units have to be taken to the site. The costs are increased also because more care is needed to handle finished units than raw materials.

(*b*) By using pre-cast concrete alone it is extremely difficult to provide continuity between the various units. Such continuity could only be achieved by using complicated mechanical connexions, a procedure which might sometimes be economically possible for heavy beams and columns but which is almost out of the question with light constructions and floor slabs.

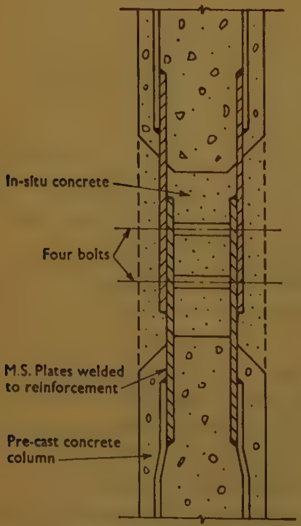
(*c*) The loss of the general monolithic effect which in-situ concrete provides. For instance, the framework for a block of flats made up of pre-cast units placed side by side could not rely upon the floor slab to provide a stiffening effect from cross-wall to cross-wall, and each frame would have to be rigid in itself; otherwise additional horizontal wind bracing would have to be introduced in the plane of the floors, whereas with in-situ construction the slab would transmit the forces without assistance.

(*d*) In many cases the greater overhead costs of a factory make pre-cast units more expensive than in-situ concrete.

One point, of supreme importance, that is not easily listed, however, either as an advantage or a disadvantage, is the question of repetition. Generally speaking, moulds for pre-cast concrete can be used far more often than ordinary shuttering, and this fact makes repetition an advantage. If there is sufficient repetition, the moulds can be made from a more expensive material, for instance steel, which gives the moulds almost unlimited life. Thus pre-cast concrete units are more economical than in-situ concrete if there is a reasonable amount of repetition and less economical if moulds have to be made for very few units. The number of units required to represent a suitable repetition differs very much with different types of units, but where the mould costs are comparatively small, the repetition need only be small. Generally, however, it can be presumed that if there is a repetition of between 50 and 100, the mould costs would be reduced to a negligible amount.

Composite construction, comprising both pre-cast units and in-situ concrete, has often been advocated as being a good compromise between the two, which retains most of the advantages of each but overcomes most of the disadvantages. The general method is to use pre-cast concrete in such a way that it forms the shuttering for the remaining in-situ concrete. If this can be done without increasing the overall dimensions, the whole of advantage (*a*) is retained without introducing disadvantages (*b*) and (*c*). It is then possible to build up a continuous construction using units and yet to retain the monolithic effect of in-situ concrete. In addition, the shape and size of the units can be influenced by this compromise, in order to simplify their transport. It will be shown later that the in-situ concrete can be used to make both slabs and beams continuous, and if necessary, to provide continuity between pre-cast beams and columns as well. It can also be used to splice two pre-cast columns which otherwise would be too long to transport in one piece. (See *Fig. 1.*)

Fig. 1



Splice of Two Pre-cast Columns

A column splice can, of course, be made mechanically, but the alternative is to leave a gap between the two pre-cast units, with reinforcing bars projecting from either side, and then complete the unit with in-situ concrete. If this joint can be housed within the confines of a beam or slab, so much the better.

The Author believes that, ultimately, composite constructions of pre-cast and in-situ concrete will be adopted almost generally in preference to other constructions, and this Paper will, therefore, first deal with the problems that arise with this type of construction and will show examples of some of the main buildings already carried out in this way. It should be emphasized here that there are no proprietary rights to composite construction. There are many patents referring to systems of floor construction, but in every case it is the shape of the components that is covered by the patent rather than the method, and this fact should allow a free and unhampered development. Later in the Paper it will be shown that the composite method has particular applications for pre-stressed concrete. If the lower part of the beam or slab only is pre-cast, it generally means that it is the tension zone which is pre-cast, and therefore, in pre-stressed concrete of the same type, only that part which is doing the tensile work can be both pre-cast and pre-stressed. This leads to improved efficiency compared with other methods by which the whole beam is pre-stressed.

The two main problems that have to be solved with composite construction are :—

(1) The adhesion between the pre-cast and the in-situ concrete.

(2) The sub-division into pre-cast units, bearing in mind the fact that, in composite construction, the units are necessarily thinner and less rigid in themselves, so that the problem of stability during transport arises.

ADHESION

In order to understand the adhesion between the pre-cast and in-situ concrete along the mating surfaces, which is usually a horizontal plane, it is necessary to recall the internal stresses to be found in an ordinary reinforced-concrete beam with particular reference to this horizontal plane. It is known from elementary mechanics that, apart from the compression and tension stresses due to bending, vertical and horizontal shear stresses occur at any point in a beam unless the shear force is zero. It is known, also, from elementary mechanics that :—

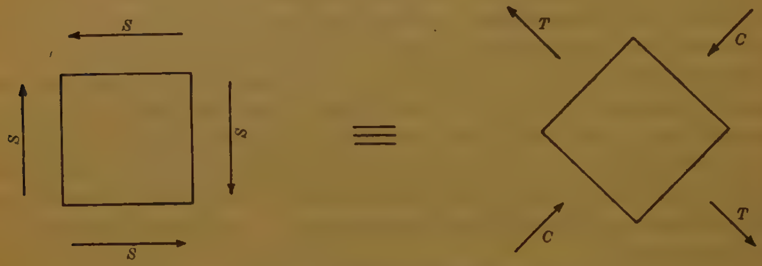
(*a*) The vertical and horizontal shear stresses are equal (see *Fig. 2*).

(*b*) Such shear stresses are equivalent to tensile and compressive stresses acting on planes at an angle to the horizontal, the largest of these stresses acting at 45 degrees to the neutral axis.

In materials like steel, which can be used in tension as well as in compression, these (principal) stresses are of comparatively little importance, but in concrete, which can take less tension than shear, destruction will often result from excessive diagonal tension rather than excessive shear.

Because, in a normal reinforced-concrete beam, the maximum diagonal tension is numerically equal to the maximum shear stress and because the concrete is so much more vulnerable to tension, only the diagonal tension is considered when calculating the dimensions of the beam. In order to simplify matters, By-laws and Codes of Practice have ceased to distinguish between diagonal tension and shear and have limited the shear values to suit the capacity of the concrete in diagonal tension. In materials which react differently to compression and tension, the ultimate shear stress can usually be taken as being approximately $\sqrt{ct}$, where c denotes the ultimate compression and t denotes the ultimate tension. For instance, if a certain type of concrete has an ultimate stress of 460 lb. per square inch in tension and 4,600 lb. per square inch in compression, the ultimate shear stress would be approximately 1,460 lb. per square inch; therefore, with a factor of safety of 4, the permitted stresses would be 115 lb. per square inch in tension, but 365 lb. per square inch in shear. In the case of ordinary beams, it is necessary, therefore, to limit the shear stress to 115 lb. per square inch also, because of the diagonal tension stress that occurs at the same time. Further, it is important to appreciate the fact that diagonal tension stresses can only occur across a plane that is diagonal, and that no tension stress or compression stress can occur across a horizontal plane in a beam.

Fig. 2



T, *C*, and *S* have the same numerical value

PRINCIPAL STRESSES

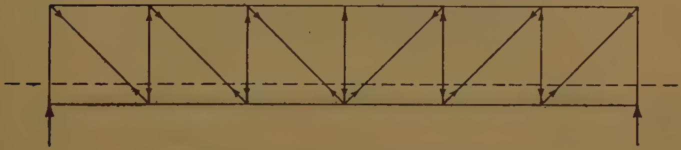
These facts have been somewhat obscured during the development of reinforced concrete, because regulations permit concrete to be reinforced in such a way that it will not disintegrate even after the concrete has cracked in diagonal tension. When the concrete cracks, the stresses re-distribute themselves completely and will adapt themselves to the resistance offered by the remaining material. For instance, if a concrete beam is reinforced so as to act as a latticed girder, tension stresses will occur after the concrete has cracked where there are tensile members, and compression stresses where there are compressive members. These compression and tension stresses will be segregated and will occur locally at alternate points along the horizontal plane. (*Fig. 3.*)

It might be said, therefore, that there are two stages :—

(1) So long as the concrete is capable of taking the diagonal tension stresses, with a reasonable factor of safety, no vertical tensile stresses occur in the beam.

(2) When the concrete is not capable of resisting the diagonal tension stresses, vertical tensile stresses may develop, which have to be taken by an adequate latticed system.

Fig. 3



Latticed Action in Concrete Beam

There are three methods by which the pre-cast and the in-situ concrete of the beam can be connected :—

(*a*) By roughening the surface of the pre-cast concrete and by pouring the in-situ concrete on to it.

(*b*) By means of projecting steelwork—stirrups or bent-up bars.

(*c*) By providing a mechanical grip in the concrete itself.

(*a*) No test results are available regarding connexion by adhesion o a rough surface, and, whilst this method is frequently suggested, it is not to be recommended unless more definite data from actual tests become available. Even with such information available, the term " rough " is so vague, that it might be dangerous to use test results as the basis for common practice unless the permitted shear stresses are assumed to be exremely low. An exception to this is the case where the shrinkage of the oncrete can be relied upon to produce the necessary bond. Shrinkage produces such a bond with round-bar reinforcement, and in some cases pre-cast units are of such a shape (see *Fig. 4*) that the in-situ concrete, when shrinking, exerts the same pressure on the pre-cast concrete as on einforcement, thus producing a reliable bond. Again, test results are not available to substantiate this, but the test described by Abeles [1] might be considered to be sufficiently conclusive.

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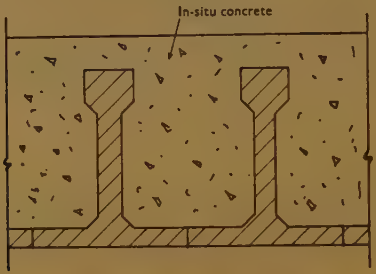
[1] The references are given on p. 243.

(*b*) If projecting steelwork is used, it is equivalent to assuming that separation may take place between the pre-cast and in-situ concrete, and that the steel reinforcement is strong enough to take all stresses which would occur. In other words, the steel in this case would have to be calculated in the same way as for ordinary reinforced concrete.

Fig. 4



PRE-CAST UNITS LAID SIDE BY SIDE FOR BRIDGE-DECK CONSTRUCTION

(*c*) Castellations can be of any of the three types shown in *Figs 5.* Quite obviously, the one in *Fig. 5* (*b*) is the best from a mechanical point of view, but the danger with this type is that the in-situ concrete might not fill the indentations completely. The type shown in *Fig. 5* (*a*) is the best practical compromise. Such castellations are easy to produce and, as tests have shown, are quite effective. The castellations shown in *Fig. 5* (*c*) are even easier to produce, but are much less effective because the diagonal tension stresses can act on the diagonal faces of the castellations. Three kinds of failure can take place in a castellated beam :—

(1) Assuming that the bond between in-situ and pre-cast concrete is non-existent and the two parts rely on the mechanical grip on the vertical surfaces of the castellations, a horizontal shear action can occur across the castellations along either line a-a or line b-b shown dotted in *Fig. 5* (*a*).

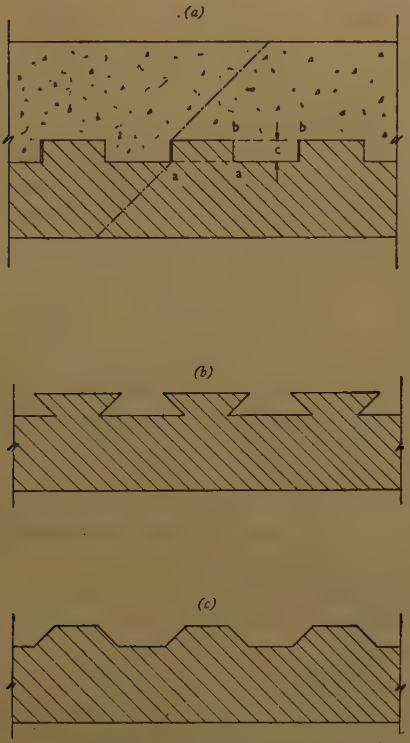
(2) If the vertical surfaces c are too small, crushing in compression could occur. This can easily be prevented by increasing the depth of the castellations to produce a greater compression area.

(3) The third cause of failure might be a diagonal tension crack, shown chain-dotted in *Fig. 5* (*a*). On the vertical faces of the mating surface, where no compression occurs, the adhesion between the two parts is very small and that part of the surface offers no resistance to shear or to diagonal tension.

The shear across line a-a or b-b is obviously twice as large as the shear in ordinary reinforced concrete, if the bond along the separating surface is assumed to be negligible.

In view of the fact, as explained earlier, that the permitted shear stress is approximately three-and-a-half-times the permitted tensile stress, it is obvious that failure in diagonal tension will take place before failure in shear across line aabb. In computing the area of resistance to shear, the depth of the castellations should be omitted from this calculation. If the depth of the castellations is y, the area of resistance against shear is approximately $b(\frac{7}{8}d - y)$. It is usually sufficient to make this depth $\frac{1}{2}$ inch.

Figs 5



Types of Castellations

These facts have been confirmed by the tests described in Appendix I. The results of both calculation and tests show that a castellated pre-cast unit can be used to replace ordinary reinforced concrete where, in accordance with calculation, the concrete is strong enough to take diagonal tension and where the shear need not be taken by steel. Where steel is needed to take the shear it would still have to be used with a castellated unit, and in this case there is very little point in having the castellations. There is no difficulty in arranging the necessary steel to connect the pre-cast and in-situ concrete even in those cases where the steel is not required to take the shear, but it means considerable extra cost and it will be found that the castellations, where applicable, are a much cheaper and equally efficient solution.

SIZE OF UNITS AND TRANSPORT

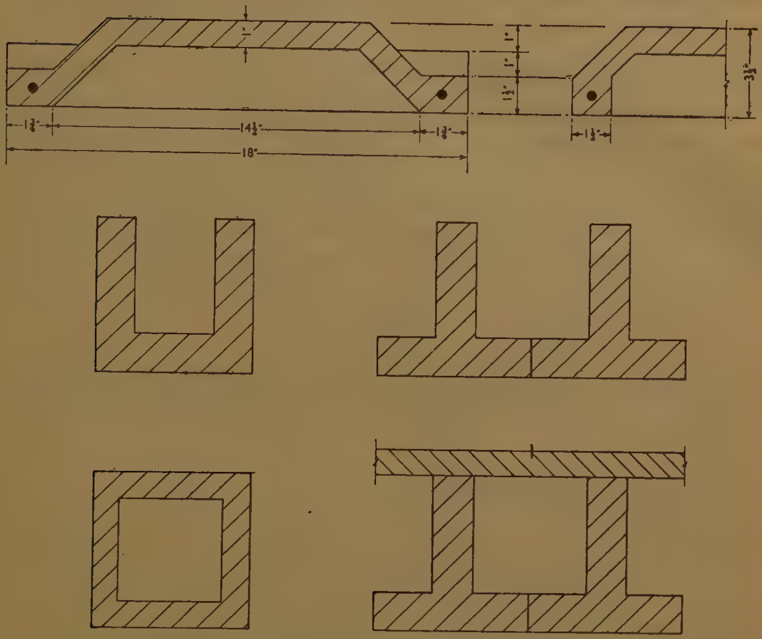
The pre-cast units in a composite construction are invariably much shallower than the full construction depth, and in fact, from an economical point of view, the tendency is to keep the units as shallow as possible. Automatically, this means a certain weakness during transport. For instance, in the case of a 4-inch slab, the pre-cast part would be only 1 inch or 2 inches thick, and whereas a 4-inch slab would be strong enough to take all stresses during transport, a slab 1 inch or 2 inches thick would have to be specially designed for transport as well as for its own structural purpose.

In general, it can be said that it is difficult to handle a unit, the thickness of which is less than one-fortieth of its largest horizontal dimension. A construction that is made up of columns, beams, and slabs will naturally be sub-divided into these components and it will not, usually, be practical to cast a column and a beam as one combined unit. Flat slabs, even 2 inches thick, cannot be made longer than about 6 feet. For this reason, special shapes have been developed, some of which are shown in *Figs 6*. The troughed units have been found to be extremely useful, particularly where a flat soffit is not required or where a false ceiling is to be introduced in any case. Such units can be 1 inch thick and derive their stiffness from their shape. These troughs become particularly rigid when the bottom lip of the trough is carried round all four sides and reinforced with a continuous-butt-welded bar. Such trough units can be made strong enough to carry, temporarily, the in-situ concrete and the concrete-barrows used during concreting, so that temporary scaffolding is not required.

Where flat soffits are required, for architectural reasons, T-, U-, or box-shaped sections are more suitable. The T- and U-sections result in a solid slab which is rather heavy, and this often makes them unsuitable for light constructions. The box-section is more expensive to produce but it gives a lighter slab and has been widely adopted for proprietary floors.

Pre-cast beams should extend up to the soffit of the floor slab only. This may make them extremely shallow, and if necessary they can be stiffened by introducing into them the stirrups for the whole beam and even the top reinforcement also. Furthermore, diagonal bars can be introduced to make a latticed girder out of the steel reinforcement, or

Figs 6



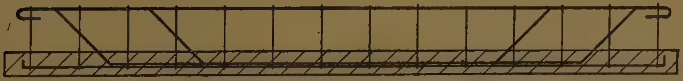
Pre-cast Units Suitable for Handling

some of the stirrups can be placed diagonally. (See *Figs 7* (*a*) and (*b*).) This will give the pre-cast section greatly increased stiffness during transport.

The stirrups can also be used to hold the pre-cast floor units in position until the in-situ concrete has been poured. For practical reasons, there

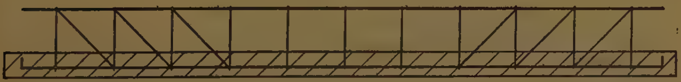
Figs 7

(*a*)



BENT-UP BARS AND STIRRUPS SPOT-WELDED TO TOP REINFORCEMENT

(*b*)



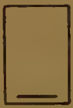
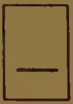
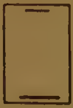
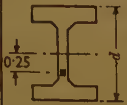
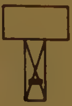
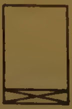
VERTICAL AND INCLINED STIRRUPS SPOT-WELDED TO TOP REINFORCEMENT

Use of Stirrups and Top Reinforcement for Stiffening during Erection should be no mechanical connexion between the floor units and the beams. Further continuity bars can be added before the in-situ concrete is poured.

The Use of Pre-stressed Concrete in Composite Construction

In simply supported pre-stressed-concrete beams, the lower part constitutes the tension zone. If this lower part is pre-cast and pre-stressed in the factory—and this can be done by any of the known methods of pre-stressing—the tension is concentrated at the place where it is most efficient, and the minimum amount of tension reinforcement is required. Table 1 compares the quantity of steel required with the cost of different

Table 1

TYPE OF BEAM		DESCRIPTION	TOTAL COST OF STEEL	EXAMPLE TAKING $\alpha = 8$, $\beta = 3$	
				COST	SAVING AGAINST R.C.
		Reinforced concrete	100	100	
A		Pre-stressing wires at lower edge of middle third	$240\frac{\beta}{\alpha}$	90	10%
C		Pre-stressing wires at lower edge of beam M.S. reinforcement at top	$37{\cdot}5 + 120\frac{\beta}{\alpha}$	82·5	17·5%
B		I-section Pre-stressing wires at 0·25*d* below centre	$160\frac{\beta}{\alpha}$	60	✱ 40%
D		Lower part of beam below neutral axis is pre-stressed independently	$111\frac{\beta}{\alpha}$	41·7	58·3%
E		Lower part of beam containing reinforcement is pre-stressed independently	$100\frac{\beta}{\alpha}$	37·5	62·5%

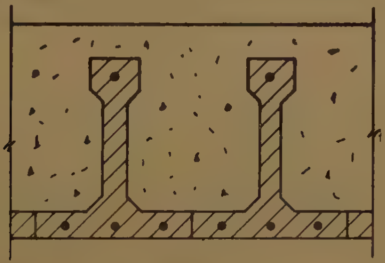
✱ Mould costs are heavier in this case

methods, and also shows that the saving in steel, by adopting pre-stressing, can be as much as 50 per cent, as compared with a non-composite pre-stressed beam; this is additional to the purely practical advantages which have already been discussed.

In Table 1 the following types of pre-stressed construction are compared, and are amplified in Appendix II, and in reference 2:—

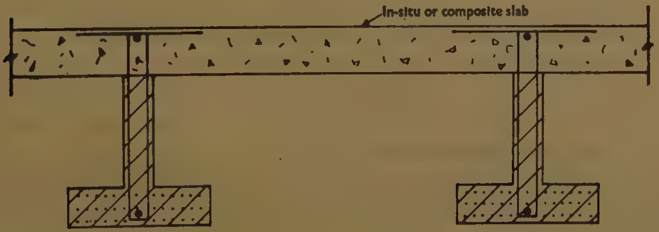
(A) Rectangular pre-stressed beams.
(B) Pre-stressed beams of I-section.
(C) Rectangular pre-stressed beams with mild-steel reinforcement at the top.
(D) Composite beams having a pre-stressed web, or T-section.
(E) Composite beams having a pre-stressed plank at the bottom.

Fig. 8



Composite Pre-stressed Concrete Construction with Pre-stressed Inverted T-Sections and In-Situ Concrete Filling

Fig. 9



Composite Pre-stressed Concrete with Pre-stressed Inverted T-Sections and Pre-cast or In-Situ Slab

The results correspond exactly with the Author's own experience, particularly for spans of between 20 feet and 40 feet, and this seems to indicate that composite pre-stressed beams are economical compared with steel and reinforced concrete, whilst non-composite pre-stressed concrete often does not appear to be economical.

In practice there seem to be three types of composite pre-stressed concrete constructions, and these are shown in *Figs 8, 9,* and *10.* The construction shown in *Fig. 8* consists of a number of pre-stressed T-sections, set side by side, to be filled in with in-situ concrete to form a slab. The webs of the tees in this case are not particularly useful for pre-stressing, but they serve the following purposes :—

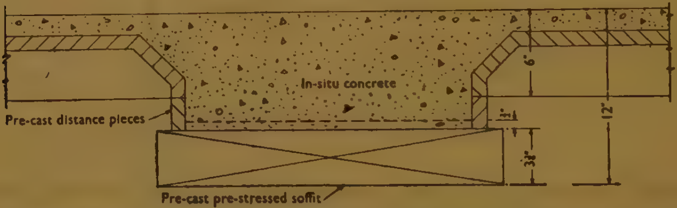
(*a*) The shear area is very much increased; and particularly by having the bulb at the top of the Tee, the bond between the pre-cast and in-situ concrete can be considerably improved since the shrinkage of the concrete will cause it to press against the bulb.

(*b*) During transport, the pre-stressed unit is stiffened by the webs.

(*c*) An increased resistance to shear is obtained in the web due to pre-stressing.

However, this last point will not be of great importance because if a horizontal section is taken through the centre of the slab, only a small part of it is pre-stressed, and the majority is ordinary concrete.

Fig. 10



COMPOSITE PRE-STRESSED CONCRETE WITH LOWER PART OF TENSILE ZONE PRE-STRESSED

This construction would appear to be suitable for bridges, etc., where the live load is so heavy that a comparatively heavy dead load is immaterial. Several constructions have been carried out on these lines, both here and on the Continent. Examples are given later in the Paper under the heading " Applications."

The construction illustrated in *Fig. 9* is carried out here and on the Continent. It can comprise either individual beams or rib-and-slab construction.

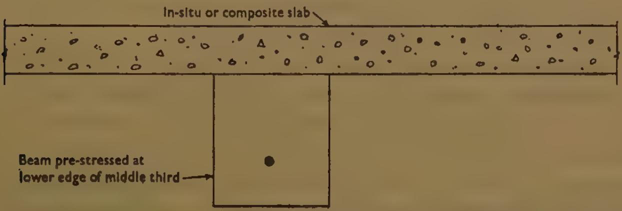
Where the ribs project below the actual slab, either a false ceiling is used, or burnt clay units are introduced to give the appearance of a solid slab, but this does not affect the principles of the construction. The main advantage here is that an economical section, resembling an I-section, is achieved, the cast-in-situ concrete forming the top slab, and the bottom flange and web being pre-stressed. For the pre-stressing of the T-section see the text referring to *Fig. 8*. Provided that there is sufficient pre-stressing in the web to take the increased shear stresses, advantages may be gained in this direction.

However, the difficulty as a rule is that economical pre-stressing is done in such a way that the stress near the top of the tee is zero or even negative, and if this is so there is no advantage to be gained for the shear stresses from the pre-stressing.

Almost invariably the connexion of the pre-stressed member to the top flange has to be effected by means of steel because the very small mating surface does not usually allow the transmission of shear stresses by the concrete. Where the section is wide enough, castellations can be introduced instead of steel, but where steel is used to transmit the shear from the web to the top slab, the same steel might as well be used as shear reinforcement in the actual web, thus cancelling out the advantages of the pre-stressing in this direction.

For small beams it is useful to pre-stress a rectangular web (see *Fig. 11*, which in principle does not differ from *Fig. 9*), but the tensile wires have to be accommodated in the web itself.

Fig. 11


In-situ or composite slab
Beam pre-stressed at lower edge of middle third

Composite Pre-stressed Concrete with Rectangular Web Pre-stressed

The third type of composite construction is one that has been used repeatedly for multi-storey buildings, often allowing the omission of intermediate columns, and thus effecting economy. The method can be used with any of the known types of pre-stressing, although tests have been carried out only on the bonded method (see Appendix IV). In the construction shown in *Fig. 10* it is only the lower part of the tensile area that is pre-stressed, and no attempt has been made to produce a pre-stressed web. The advantages of this construction are particularly apparent where it is necessary to introduce a wide but shallow beam. It should be noted here that such a wide shallow beam in pre-stressed concrete is often more economical than a deeper beam, whereas in ordinary reinforced concrete the opposite would be the case.

Owing to the great width of the pre-stressed soffit, the shear stresses are rarely above those permitted in ordinary reinforced concrete, so that a beam, with castellations, can be employed. Where necessary, of course, stirrups or bent-up bars can be cast into pre-stressed units, projecting up into the in-situ part of the structure.

This method has been used extensively for floor constructions in

Switzerland and in France. Examples are given later in the Paper under the heading " Applications."

In Great Britain it was found possible to develop and produce three standard pre-stressed units which could be employed under conditions where different bending moments arise. Two units were 3¾ inches deep, and the third was 3¼ inches deep, and all varied in width and steel content. For their standard calculations, see Appendix III, which also shows typical applications for various bending moments. Some very interesting results have been obtained from a loading test carried out at the laboratory of Messrs R. H. Stanger on a T-beam. The beam was made up of a pre-stressed flange, reinforced with sixty 10-gauge wires, and in-situ concrete to form the web and complete the top part of the flange. (See *Fig. 43* in Appendix IV, where the test is described.) The test showed again that castellations, between the pre-stressed soffit and the in-situ concrete, work efficiently.

As can be seen from the test results, final failure took place at almost exactly the calculated stress. Also, the first hair cracks appeared in the unit when, according to calculation, the pre-stress was a little more than offset by the tension in the bottom due to load.

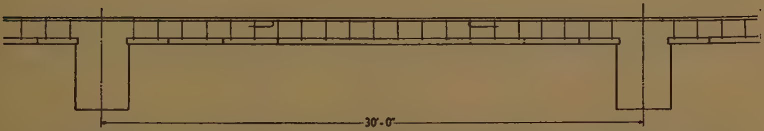
This indicates that the method of calculation is representative not only of the actual stresses that occur, but also of the factor of safety. In this respect the composite construction varies considerably from the usual method using pre-stressed concrete, when the whole of the beam is pre-stressed. With the latter method the neutral axis moves upwards from the bottom edge if the load is increased beyond the design load, with the result that the steel stress is reduced and the concrete stress is increased, as compared with the calculated stresses assuming the neutral axis to remain at the bottom edge of the beam. This provides a hidden factor of safety (so far as the steel is concerned), which is not there if only the tension zone is pre-stressed. On the other hand, with the usual pre-stressed construction, the factor of safety on the compression is reduced, but it remains the same when only the tension zone is pre-stressed, until the stress in the steel exceeds substantially the limit of proportionality (for practical purposes 0·1 per cent proof stress). After that, of course, the neutral axis must rise in any case, as in ordinary reinforced concrete. Even then, the rise in the composite construction is very much smaller than in ordinary construction, with the result that final failure in a composite beam (with only the tension zone pre-stressed) will take place first in the steel; whilst with the more usual type of pre-stressed concrete, it will take place first in the concrete. This was confirmed in the test, since the cracks reached the top flange but did not penetrate into it, even at failure.

The test beam showed 97-per-cent recovery after 114 per cent of the design load had been applied. It still showed a recovery of 95·6 per cent after the steel had been stressed to 98 tons per square inch—6 per cent above the guaranteed 0·1-per-cent proof stress.

From this, it appears that in calculating the factor of safety for this type of composite construction, it is less important to refer to the proof stress than to the ultimate stress, and it is suggested that the design stress in the reinforcement be limited to $\frac{1}{1{\cdot}6}$ of the proof stress or to $\frac{1}{2{\cdot}25}$ of the ultimate stress of the reinforcement, whichever is the smaller.

One special application of the third type of pre-stressed unit can be found in continuous construction. It would be possible, of course, to use pre-stressing merely for the positive moment, while the negative moment was taken by ordinary mild-steel reinforcement. (See *Fig. 12.*) This would not be very logical, particularly as the negative moment is usually larger than the positive moment, and a method has been evolved by which similar units to the pre-stressed soffits are used as top reinforcement for the negative moment only. Such a construction is shown in *Fig. 13* (*a*).

Fig. 12



Continuous Beam with Negative Moment taken by Mild-Steel Reinforcement

A special problem to be dealt with is that the pre-stressed soffit would be submitted to additional compression stresses near the end where the negative moments occur. To counteract this, a combined soffit was produced (see *Fig. 13* (*b*)), consisting of shorter pre-stressed units, made up with ordinary concrete near the ends so that the centre part of the beam, between the points of contraflexure, is pre-stressed, but the parts which may receive additional compression are not. The problem is simplified by using several pre-stressed soffits side by side, staggered against each other to allow for a change in bending moment which might cause the points of contraflexure to move. This is similar to the curtailing of bars in ordinary reinforced concrete.

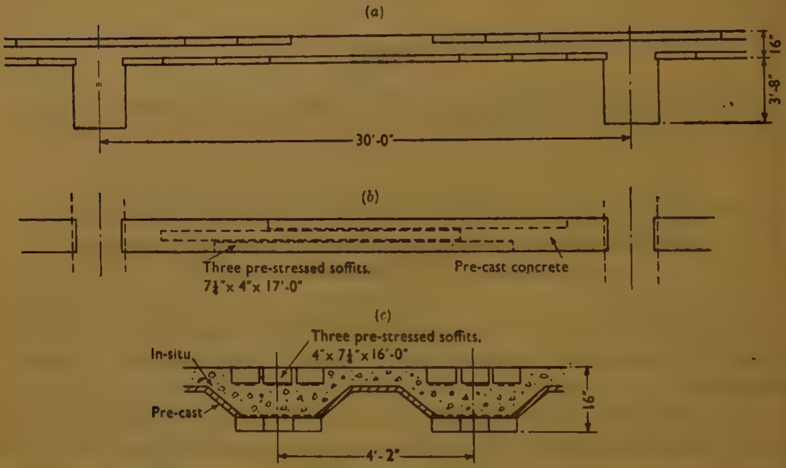
One general point should be noted, which is borne out by the calculations in Appendix III. When using pre-stressed concrete together with ordinary reinforced concrete, the moduli of elasticity are not the same and this fact must be taken into account for realistic calculations of deflexions.

It is well known that, for good concrete, the modulus of elasticity immediately after applying the load is in the neighbourhood of 6,000,000 lb. per square inch, but under a load which remains steady for a considerable time the apparent modulus of elasticity is very much smaller owing to creep.

It would often be correct to calculate the deflexion due to live load using a modulus of elasticity of 6,000,000 lb. per square inch, and the deflexion due to dead load using a modulus of elasticity of 1,500,000 lb. per square inch.

However, in pre-stressed concrete the situation is considerably different because the creep in the pre-stressed member acts against the bending moment. In a beam with a pre-stressed soffit which is stressed by the dead load to half its capacity, there is still compression in the soffit, tending to shorten the beam at any time, and not to lengthen it. The apparent modulus of elasticity of this bottom part tends to be very much higher than 6,000,000 lb. per square inch; it may even be infinite and it may even be negative. In other words, it is quite possible that a pre-stressed beam has an upward deflexion due to creep during the period when only dead load acts, and this has been proved in practice. To incorporate this in the calculation, a reasonable knowledge of creep should be available, and this is complicated by the fact that the creep really depends on the age of the concrete when the unit is pre-stressed and the cast-in-situ concrete poured.

Figs 13



CONTINUOUS BEAM WITH PRE-STRESSED SOFFITS USED TO TAKE THE NEGATIVE AND POSITIVE BENDING MOMENTS

If there is any doubt about the creep, a modulus of elasticity of 6,000,000 lb. per square inch can be used for the pre-stressed part, which would definitely be on the safe side. For the in-situ part it would then be reasonable to use the values given for dead and live load.

APPLICATIONS

There are a number of proprietary floors on the market which make use of composite construction, some of which are Milbank, Myko, Stahlton, Triad, Poutrelle Prima-Rex, and other French floors. Of these, the ones making the most use of composite construction are those in which the beams, as well as the slabs, are composite, such as the Poutrelle Prima-Rex floor which is shown in *Fig. 14.* *Fig. 15* shows beams and slabs at the Hatfield Technical College during construction. The beams here are trussed beams with a pre-cast concrete upper chord, pre-cast concrete struts, and a mild-steel-flat lower chord. The trussed beam is simpler to mould than a latticed construction, yet it retains the advantages of latticed construction and, at the same time, any bending moment arising from one-sided load can easily be taken by the upper chord. With these particular beams the upper chord is 7½ inches deep, but the floor slab consists of troughs with 1½-inch topping. The aggregate thickness of the floor is 4½ inches so that only the lower 3 inches of the beam is pre-cast. The beam is 7 inches wide. As can be seen from *Fig. 15,* the stirrups and top bars give the units added stiffness during transport.

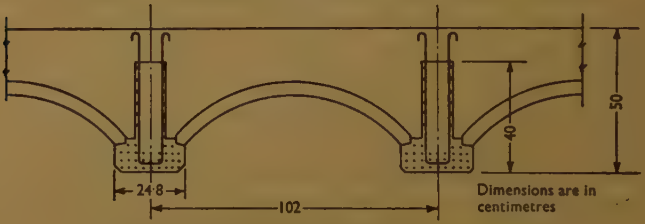
The trough units, used for the floor slab, were calculated to take the dead weight of the in-situ concrete, acting as beams on two supports, and spanning between the beams. They serve, as part of the finished floor slab, to take the live load, and the finished floor slab, comprising pre-cast troughs and in-situ concrete topping, is continuous. All the positive steel for the floor slab is in the troughs, the negative steel for continuity being added in the in-situ concrete. *Fig. 16* shows the connexion between a trussed beam and a column. A slot is arranged in the upstanding part of the tie member which fits over a plate projecting from the column. Square wedges are driven into the two holes to take both the vertical force and the bending moment caused by wind forces. The whole connexion is completely covered afterwards by the in-situ concrete; in this manner a sufficiently tight fit between steel and concrete is guaranteed to ensure the transmission of stresses from one to the other.

Fig. 17 shows a system of main and secondary beams, consisting of pre-cast concrete arches, which is used for a factory for Messrs Colodense Ltd, at Bristol. The main arch spans between columns 33 feet 4 inches apart, and in turn carries secondary arches of similar type. Floor troughs of the type already described span between the upper chords of the secondary arches. Again, only 3 inches of the upper chords of the arches were pre-cast, the remainder being part of the in-situ topping. As with the Hatfield College, the in-situ concrete covers all connecting bolts, etc.

Pre-stressed Concrete

Fig. 18 shows a series of pre-stressed units, T-shaped, with bulbous heads, laid side by side to form the structure of a railway bridge at Gilroyd Lane on the Worsborough Branch. The units are afterwards covered with solid concrete with nominal reinforcement for a thickness of 4 inches. Details of this and other bridges have been given by P. W. Abeles.[1, 3, 4] The Eastern Region of British Railways has adopted this type of construction extensively. Similar bridges have also been carried out on the Continent, notably in Germany and near Gram in Denmark.[5]

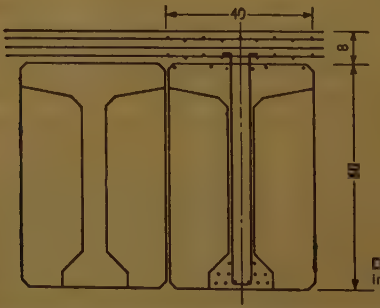
Fig. 19



Section through Roof of a Coal Bunker

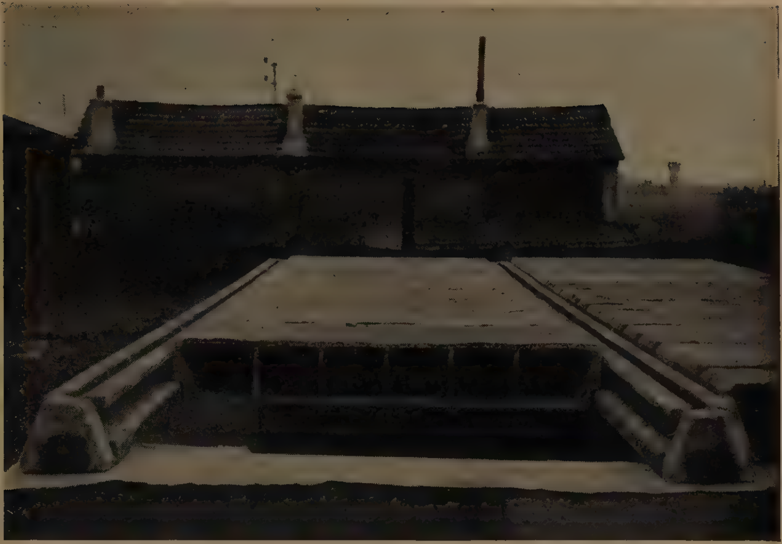
There are several variations of this type of construction, for example, the cross-section shown in *Fig. 19* has been used on several occasions in Switzerland.[6] In the type shown in *Fig. 18*, a rather thick slab is suitable only in cases where there are such heavy live loads that the dead load becomes negligible in proportion. The construction shown in *Fig. 19*, on the other hand, has less weight in the slab and is more suitable for ordinary floors. Other examples, calculations for constructions of this type, and

Fig. 20



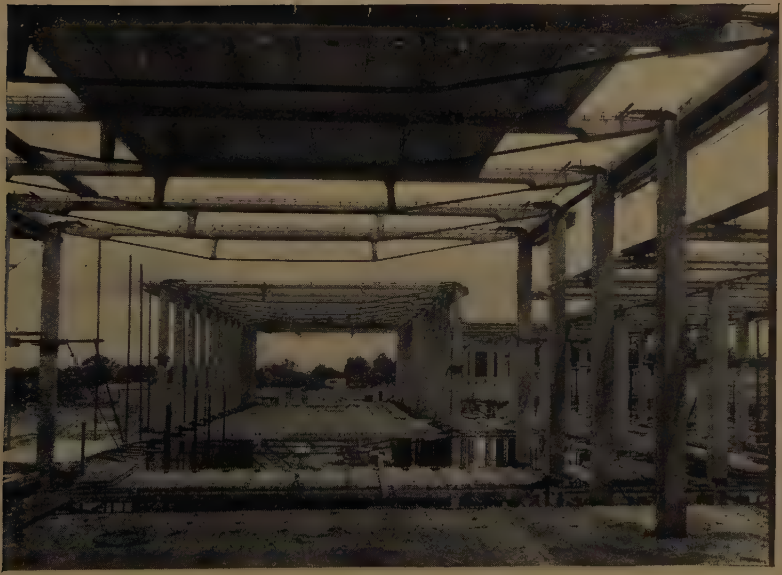
Section through Avalanche Gallery (Swiss Railways)

Fig. 14



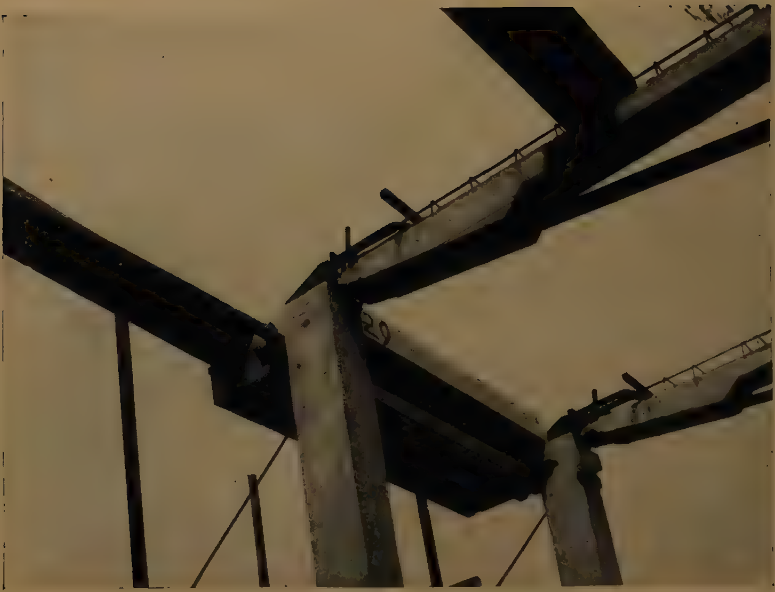
Prima-Rex Floor Construction

Fig. 15



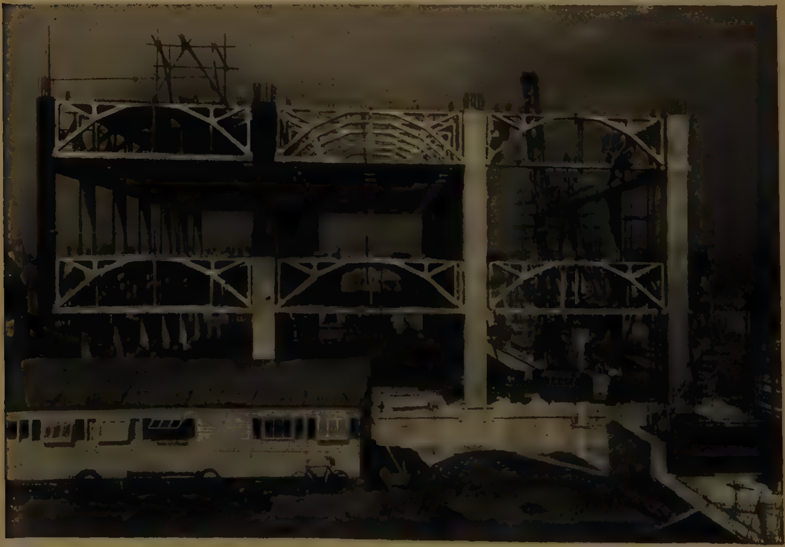
Hatfield Technical College during Construction showing Trussed Beams and Trough Units

Fig. 16



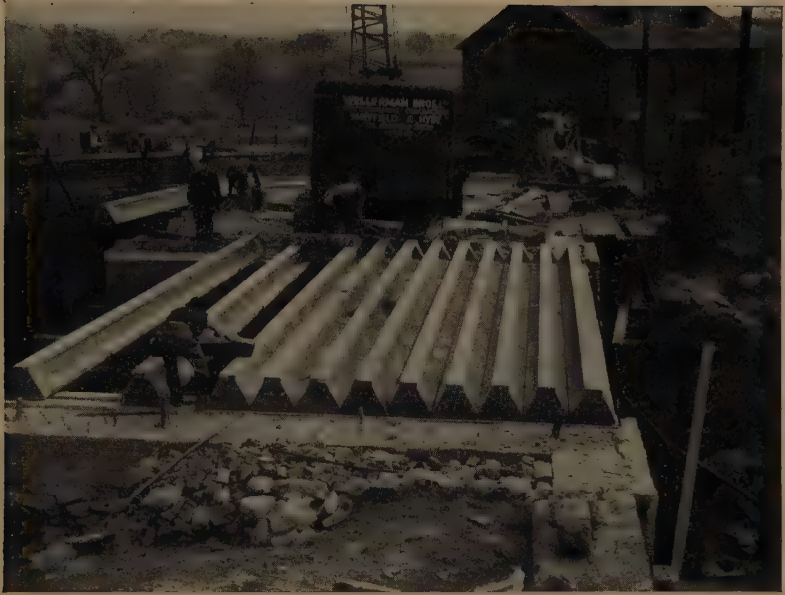
Hatfield Technical College. Connexion between a Trussed Beam and Pre-cast Columns

Fig. 17



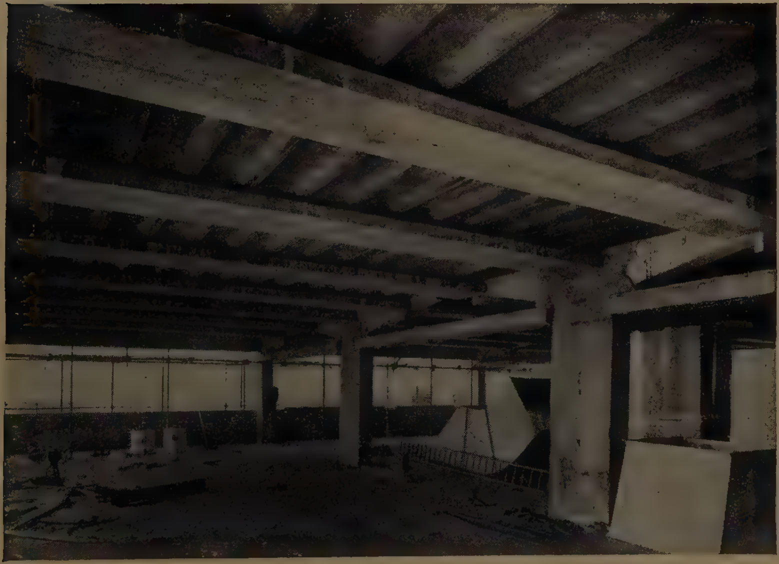
Factory at Bristol showing Pre-cast Concrete Arches used as Main and Secondary Beams

Fig. 18



Railway Bridge at Gilroyd Lane using T-shaped Pre-stressed Units

Fig. 21

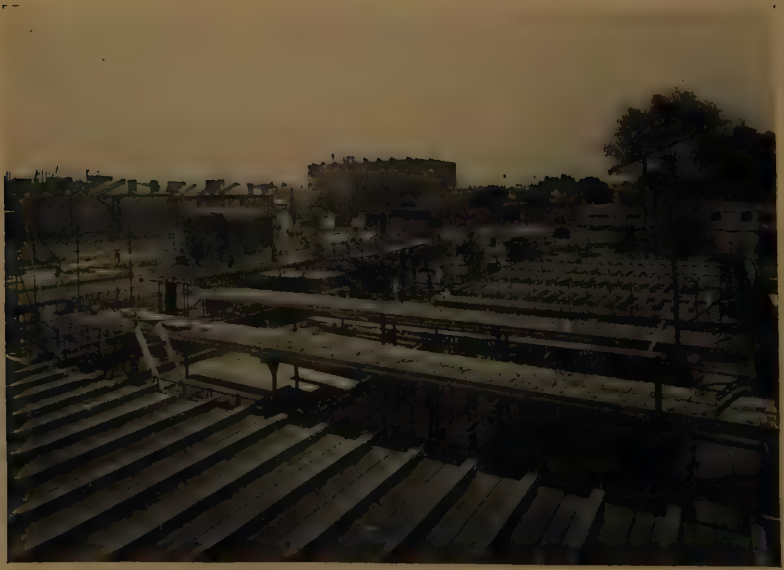


Pre-stressed Composite Beam used for a Factory at Bristol

Fig. 22

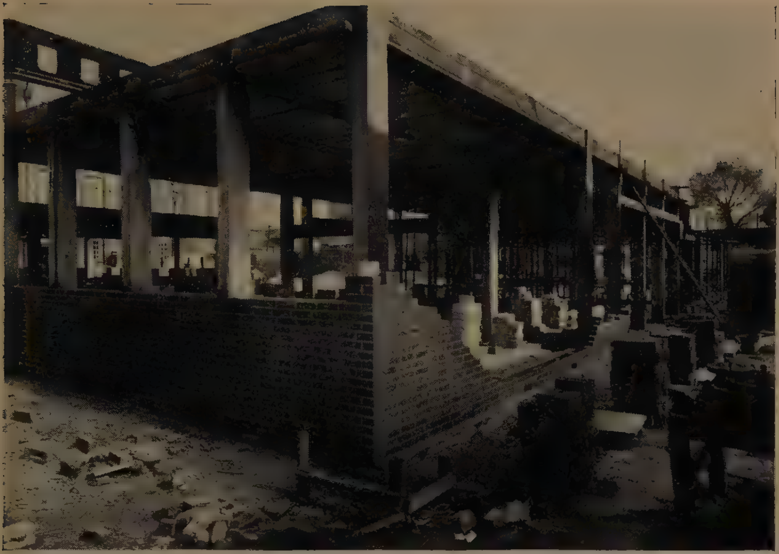
Pre-stressed Composite Beam for an Assembly Hall of a School in East London

Fig. 23



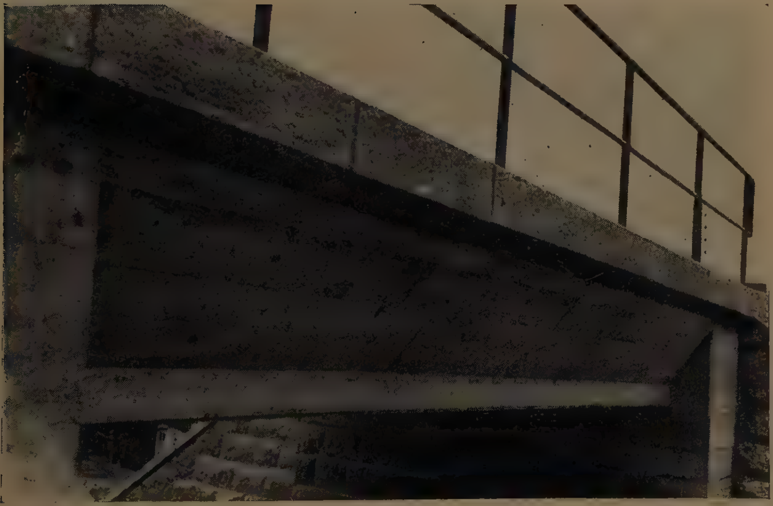
Construction of a School Dining Hall, showing Pre-stressed Soffit and Pre-cast Inverted Tees before Placing of Woodwool Slabs and In-Situ Concrete

Fig. 24



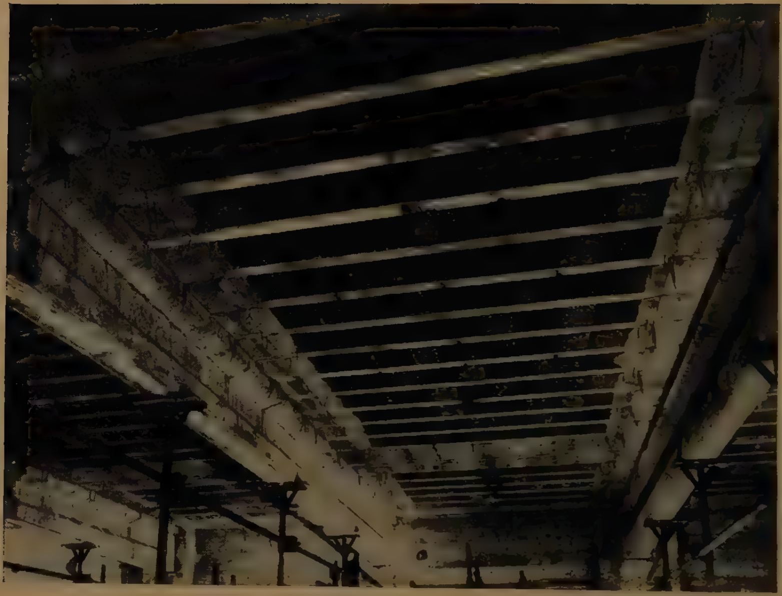
Dining-Hall Construction showing Flat Ceiling

Fig. 25



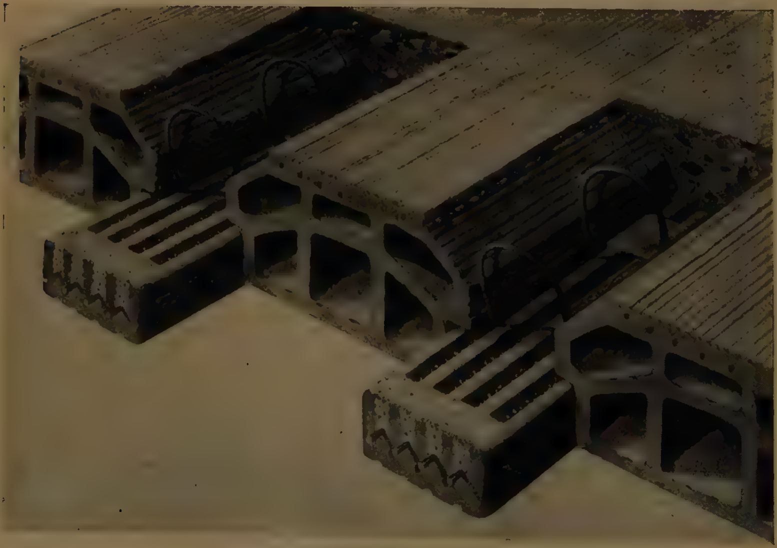
Small-Span Bridge in Switzerland using Pre-stressed Soffit

Fig. 26



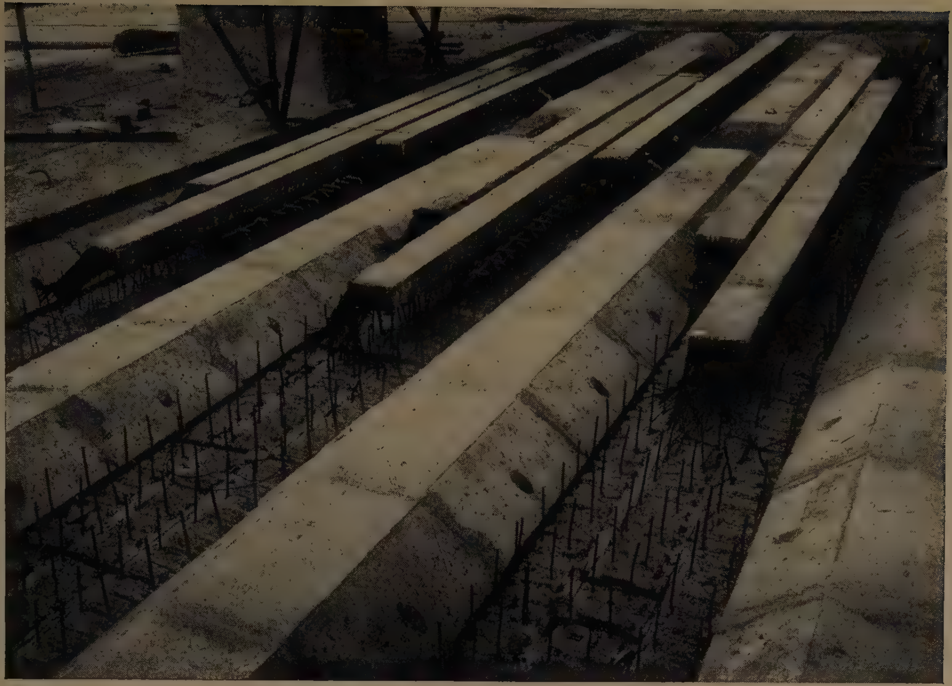
Pre-stressed Beams (Magnel Method) used for Queen Mary College, London

Fig. 27



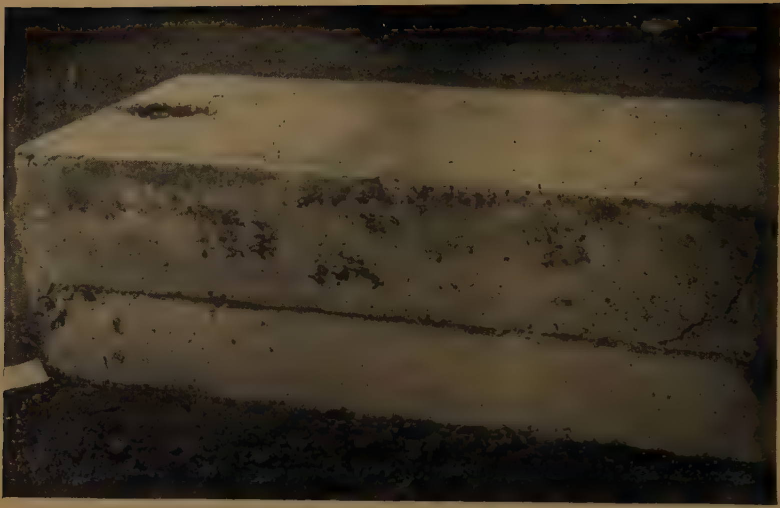
Stahlton Floor Construction

Fig. 28



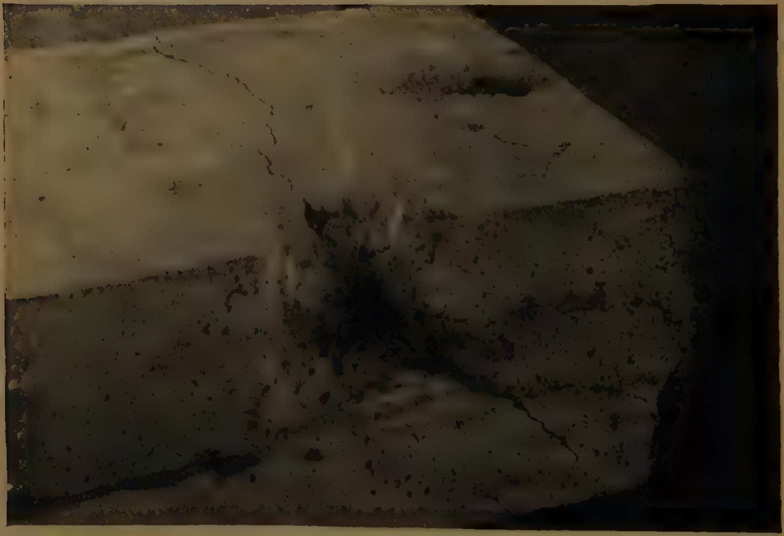
Continuous Composite Floor Construction for a Factory at Bristol, showing Placing of Pre-stressed Soffits to take Negative Bending Moments at Supports

Fig. 31



Diagonal and Horizontal Cracks in Beam Type 1

Fig. 32



Some Concrete Removed from Side of Beam Type 1, showing that Penetration of Horizontal Crack was only 1 Inch details of tests are given by P. Soutter [7] and Dr P. Lardy [8] of Zurich; a characteristic example is the gallery against avalanches for the Swiss Railways, shown in *Fig. 20.*

Castellated pre-stressed planks are particularly useful for multi-storey buildings where the headroom is very restricted. *Fig. 21* shows beams of 27-feet span used for the above-mentioned factory at Bristol. The overall depth of the beams is 12 inches, the thickness of the pre-stressed plank being 4 inches and, since the total thickness of the troughs and in-situ concrete forming the slab is 5½ inches, this left a gap of 2½ inches between the plank and the soffit of the floor slab. This 2½ inches was made up by distance pieces on which the floor troughs rested. The in-situ concrete finished-off both beams and slab. For a section of this beam see *Fig. 10.*

Additional pre-stressing in the plank was achieved by giving it an initial upward camber, that is to say, converting a uniformly distributed tension stress into one which went from zero at the top to the maximum at the bottom.

The pre-stressing wires were arranged accordingly so that their centre of gravity coincided with the third point.

A pre-stressed beam of this type, spanning 32 feet, which was used for the assembly hall of a school in East London, is shown in *Fig. 22.* In this case secondary beams span between the main beams, to which plaster-board is fixed on the underside to provide a false ceiling. The secondary beams carry woodwool slabs on top of which in-situ concrete is cast.

Again, the in-situ concrete forms the upper part of the beam with the pre-stressed plank, and distance pieces were used to fill the gap between the woodwool slabs and the pre-stressed planks.

Another application of the same method has been used for the dining hall in another school and is shown in *Fig. 23.* The main construction is the same but the secondary beams have been made from inverted tees covered with woodwool slabs and in-situ concrete, so that a flat ceiling was obtained, as shown in *Fig. 24.*

Pre-stressed soffits have also been used for small-span bridges in Switzerland,[6] as shown in *Fig. 25* (note the castellations).

Fig. 26 shows beams for the Queen Mary College in London. The beams were pre-cast up to the underside of the floor slab and then post-tensioned in position. The floor slab was cast after the post-tensioning. In this way some of the practical advantages of composite construction were combined with the advantages of pre-stressed concrete. One type of floor that has not yet been carried out in Great Britain, but which is very popular in France and Switzerland, is the Stahlton floor[9], which works on the same principle as the pre-stressed planks. With the Stahlton floor, however, which is shown in *Fig. 27,* clay units with ribbed tops are laid out end to end, wires are inserted between the ribs, and the line of units is pre-stressed. The area around the wires is then filled with concrete.

In this way a " plank " is produced, not from concrete but from tiles. Hollow tiles are then laid between these " planks " and in-situ concrete poured on top. In this case all the shear stresses are taken by mild-steel stirrups which project from the tiles into the in-situ concrete. (The photograph is slightly misleading; the in-situ concrete actually covers the hollow blocks.)

Figs 13 show a continuous floor, constructed for a superload of 600 lb. per square foot on a 30-foot span. Ribs were arranged at 4-foot-2-inch centres, each rib being made up from a pre-cast plank, with in-situ concrete on top. Each plank consists of three pre-stressed units, laid side by side, staggered against each other, so that the positive bending moment is well covered in the centre, but beyond the points of contraflexure only a part of the cross-section is pre-stressed.

In this way, overstressing of the concrete, caused by additional compression at the bottom resulting from the bending moment, is avoided. For the negative moment over the supports, three single pre-stressed units were used, side by side, very much in the manner of reinforcing bars, as shown in *Fig. 28.* These units were spaced a little distance apart so that concreting could be carried out between them.

The pre-stressed units were stressed by the Hoyer method and the wires were left projecting each end to ensure a good bond between pre-stressed units and in-situ concrete. Non-reinforced pre-cast arched units formed the side shuttering for the ribs and also the shuttering for the floor slab.

One application of composite construction, which may assume greater importance in the future, is in shell construction, particularly for cylindrical or prismoidal shells. With these constructions, there is severe tensile stress at the bottom which extends over only a very small part of the section. In a number of cases, it is useful to pre-cast and pre-stress only this part, that is, the part of the structure that looks like a projecting beam. The remainder of the constructions can then be carried out in-situ. An example of this is in the hangar constructed at Karachi.[10]

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E. S. and A. Robinson, Ltd	for *Figs 21, 28.*
Holland & Hannen and Cubitts, Ltd. . . .	for *Figs 17, 23, 24.*

Stressed Concrete Design, Ltd	for *Fig. 26.*
Ministry of Works	for *Fig. 25.*
S.A.T.N.E.C., Paris	for *Fig. 14.*
Vorgespannter Beton	for *Figs 19, 20.*

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The Paper is accompanied by twenty photographs and twenty-three sheets of diagrams, from which the half-tone page plates, and the figures in the text have been prepared, and by the following four Appendices.

APPENDIX I

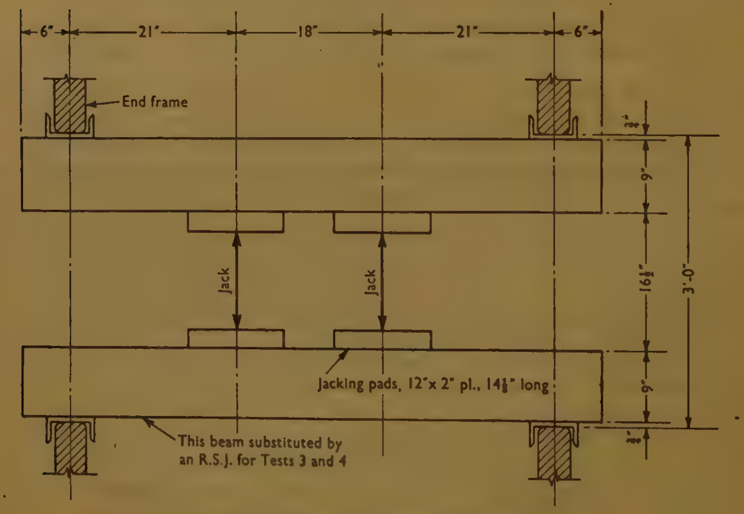
REPORT ON THE SHEAR TESTS CARRIED OUT AT THE WORKS OF THE LIVERPOOL ARTIFICIAL STONE CO., LTD

The tests were undertaken in order to determine the strength of various composite beams listed below, both absolute and when compared with a reinforced-concrete beam of the same overall section.

In order that the beam should fail in shear before failure due to bending occurred, it was necessary to test comparatively short beams and a length of 5 feet between centres of supports was selected, with two point loads each 21 inches from the supports.

The point loads were distributed over a length of 12 inches in order that the maximum shear and the maximum bending moment should not coincide, and to produce an arrangement more similar to uniformly distributed loading.

Fig. 29


6"
21"
18"
21"
6"
End frame
Jack
Jack
Jacking pads, 12"x 2" pl., 14½" long
This beam substituted by an R.S.J. for Tests 3 and 4
9"
16½"
9"
3'-0"

ARRANGEMENT FOR SHEAR TESTS

For beams Types 1 and 2, a pair of beams of each type were tested together. They were supported at the ends by two end frames and jacks were introduced between them at the required points to produce the loads as shown on *Fig. 29*. For beams Types 3 and 4, it was found easier to test one beam of each type with a rolled steel joist.

Four types were cast, as shown in *Figs 30*. Each beam had an overall depth of 9 inches and a width of 15 inches and was 6 feet long.

Beam Type 1.—Composite beam made up of a pre-stressed soffit and in-situ topping. The soffit was 15 inches wide and 3¾ inches deep, excluding castellations which were ½ inch high and 13 inches wide, so that there was a strip 1 inch wide on each side without castellations. The soffit was reinforced with sixty wires of 10 gauge, stressed to 58 tons per square inch.

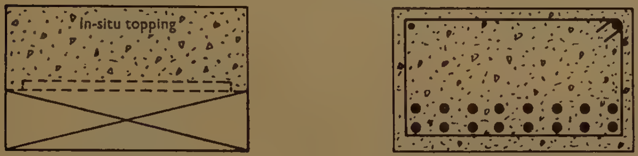
Beam Type 2.—Normal reinforced-concrete beam, reinforced with sixteen ⅝-inch-diameter bars, the centre of gravity of which was $1\frac{15}{16}$ inch from the bottom of the beam so that this almost coincided with the centre of the wires in the pre-stressed soffit.

Beam Type 3.—Composite beam similar to Type 1, but with the castellations on the pre-stressed soffit extending across the whole width of the beam (that is, 15 inches wide).

Beam Type 4.—Composite beam made up of a pre-cast reinforced-concrete soffit, 15 inches wide and 3¾ inches deep, reinforced with sixteen ⅝-inch-diameter bars and with castellations 15 inches wide.

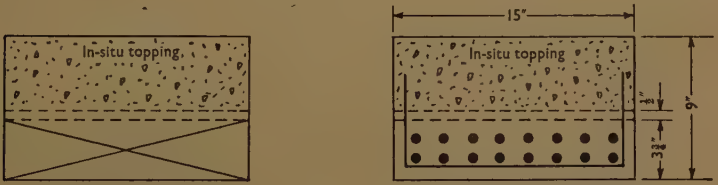
The reinforced concrete, both for the ordinary beams and the topping of the pre-stressed units was mix 1 : 2 : 4, non-vibrated, with ¾-inch aggregate and 2-inch slump. Beams Types 1 and 2 were tested when 19 days old and the compressive strength of concrete, tested at 21 days, was 2,820 lb. per square inch. Beams Types 3 and 4 were tested when 14 days old.

Figs 30



Pre-stressed soffit
Castellations: 13″ wide x ½″ deep
Reinforcement: Sixty 10-ga. wires
BEAM TYPE 1

Normal reinforced-concrete beam
Reinforcement: Sixteen ⅝″-dia. bars
BEAM TYPE 2



Pre-stressed soffit
Castellations: 15″ wide x ½″ deep
Reinforcement: Sixty 10-ga. wires
BEAM TYPE 3

Pre-cast reinforced-concrete soffit
Castellations: 15″ wide x ½″ deep
Reinforcement: Sixteen ⅝″-dia. bars
BEAM TYPE 4

BEAMS FOR SHEAR TESTS

Full details of the tests are given in Tables 2 to 5 (see pp. 247–250).

OBSERVATIONS

The reinforced-concrete beam Type 2 failed due to diagonal tension at about 430 lb. per square inch, giving a factor of safety against cracking of $\frac{430}{100}$ (4·3). (The design shear stress for a concrete mix 1 : 2 : 4 is given in Codes of Practice No. 114 as 100 lb. per square inch.) The result was very much in accordance with common experience.

Beam Type 1 ultimately failed due to diagonal tension. A horizontal crack appeared between the in-situ concrete and pre-stressed soffit and was found to penetrate 1 inch into the beam where there were no castellations. When testing beam Type 3, which had castellations over the whole width of the beam, there was no trace of horizontal cracks, and the beam failed in diagonal tension at 45 degrees over the whole depth of the beam in a similar way to the reinforced-concrete beam.

On testing the first beam Type 3, a bond failure occurred with a load of 10 tons on the jacks and, after this, the beam then failed in diagonal tension. Because of this, the results were disregarded and another beam was tested.

With the second beam Type 3, no bond failure took place and the beam failed in diagonal tension. The latter test compares favourably with the test on beam Type 1.

Beam Type 4, the composite beam made up of a pre-cast reinforced-concrete soffit and in-situ topping, failed in diagonal tension at 45 degrees. The diagonal tension crack was continuous through the castellations at 45 degrees and there was no sign of horizontal cracking between the pre-cast soffit and the in-situ concrete.

It is reasonable, therefore, when calculating the shear stress, to take the width of the area taking shear as the width of the castellations. On alternate vertical surfaces of the castellations there is no compression and little adhesion, so that the sectional area of the castellations cannot be considered to be effective in taking the shear. The depth, then, of the area taking shear must be considered equal to the lever arm minus the depth of the castellation.

Using these facts, the calculated diagonal tension at failure for beam Type 1 was 586 lb. per square inch, for beam Type 3, 477 lb. per square inch and for beam Type 4, 415 lb. per square inch. These results compare favourably with the 430 lb. per square inch for the reinforced-concrete beam Type 2.

Allowing for the fact that the castellations permit only half the length of the beam to be in shear (neglecting the bond between the in-situ topping and pre-cast soffit) the shear stress on the castellations must equal twice the diagonal tension. Since no horizontal shear failure occurred, the ultimate shear stress must be greater than twice the diagonal tension.

These tests confirmed the well-known fact that concrete can take considerably more shear than diagonal tension. The usual assumption is that the ultimate shear stress is $\sqrt{ct}$, where c is the ultimate compressive stress and t the ultimate tension stress in concrete.

Assuming a tensile stress of 400 lb. per square inch and a compressive stress of 2,800 lb. per square inch, that would give a calculated ultimate shear stress of 1,060 lb. per square inch. In beams Types 1, 3, and 4 the shear stresses on the castellations at failure, due to diagonal tension, were 1,172 lb. per square inch, 954 lb. per square inch, and 830 lb. per square inch respectively.

It is, therefore, necessary to distinguish between diagonal tension and shear whenever these two are not equal. They are equal for an ordinary concrete beam, but not for a castellated beam. In a beam with a castellated soffit the diagonal tension is completely independent of the castellations and cracks appear at approximately the same stress both in ordinary beam and a composite beam. The permitted shear stress in a castellated composite beam, however, should be at least twice the permitted diagonal tension.

CONCLUSIONS

1. All the composite beams failed in diagonal tension at stresses ranging from 415 lb. per square inch to 586 lb. per square inch. There was no sign of any horizontal cracks even at the castellations. It can be concluded that a castellated composite beam acts like a normal reinforced beam and has a similar factor of safety against cracking.

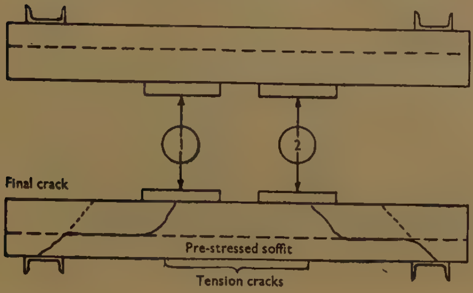
2. When calculating the shear stress in a castellated composite beam, the area of shear resistance must be taken as :

width of castellation × lever-arm depth of castellation.

3. The permitted shear stresses in a castellated composite beam should be at least twice the permitted diagonal tension.

TABLE 2

Beam Type 1. (Tested at age of 19 days)



In-Situ Topping

Mix: 1 : 2 : 4 unvibrated.
Strength at 21 days: 2,820 lb. per square inch.

Pre-stressed Soffit

15″ × 3¾″ with castellations.
13″ wide × ½″ deep.
Sixty 10-gauge high-tensile wires.
Area of steel = 0·772 square inch.

Shear Resistance

Permissible shear stress = 100 lb. per square inch.
Width of area taking shear = width of castellations = 13 inches.
Depth of area taking shear = lever arm − depth of castellation
= 5·51 = 0·50 = 5·01 inches.

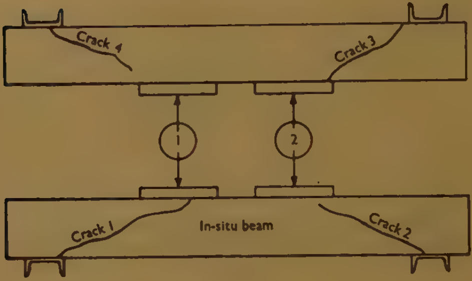
$$\frac{\text{Shear force}}{13 \times 5{\cdot}01} = \text{diagonal tension at supports.}$$

Test Results

Load: tons		Shear stress: lb./sq. in.	Remarks
1	2		
14	14	483	Tension cracks in pre-stressed soffit at centre of beam 1.
17	17	586	Diagonal tension and horizontal shear cracks in beam 1 as shown in *Fig. 31* (facing p. 241). Jacks dropped back to 15 tons.
18	18	621	Diagonal tension cracks penetrated whole depth. Impossible to increase load on jacks.
0	0	0	On unloading, horizontal crack was found to penetrate only 1″. See *Fig. 32* (facing p. 241).

TABLE 3

Beam Type 2



In-Situ Concrete

Tested at age of 19 days.
Mix : 1 : 2 : 4 unvibrated.
Strength at 21 days : 2,820 lb. per square inch.
Top steel : two ⅜"-diameter bars.
Bottom steel : sixteen ⅝"-diameter bars.
Stirrups : ¼"-diameter at 6" centres.

Shear Resistance

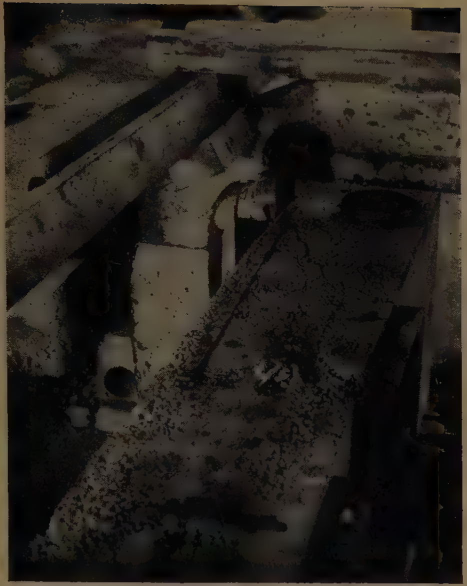
Permissible shear stress = 100 lb. per square inch.
Area taking shear = width of beam × lever arm = 15 × 5·52 square inches.

$$\text{Shear stress} = \frac{\text{shear force}}{15 \times 5{\cdot}52}$$

Test Results

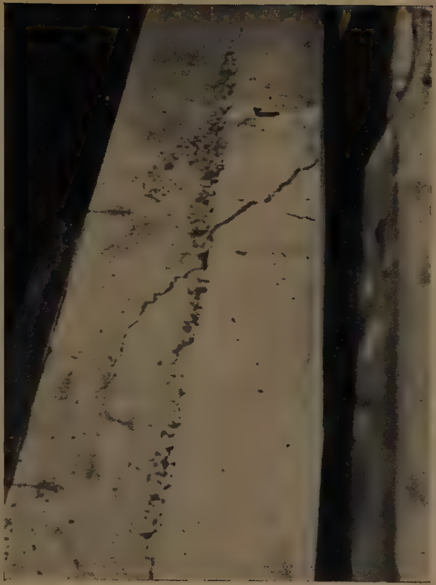
Load : tons		Shear stress : lb./sq. in.	Remarks
1	2		
12	12	325	Crack heard.
16	16	432	Diagonal tension crack No. 1 appeared as shown in *Fig. 33*, and jacks dropped back to 15 tons.
18	18	486	Cracks Nos 2, 3, 4 appeared.

Fig. 33



Diagonal Tension Crack in Beam Type 2

Fig. 34

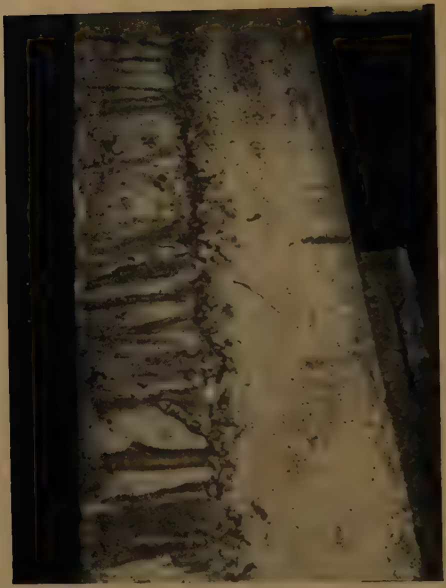


Tension Crack in Pre-stressed Soffit and Diagonal Tension Crack in Beam Type 3

Fig. 35

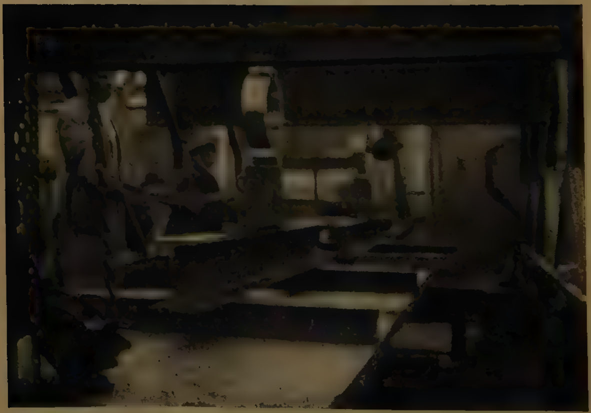
Another View of Tension Crack in Pre-stressed Soffit, Beam Type 3

Fig. 36



Diagonal Tension Crack in Beam Type 4

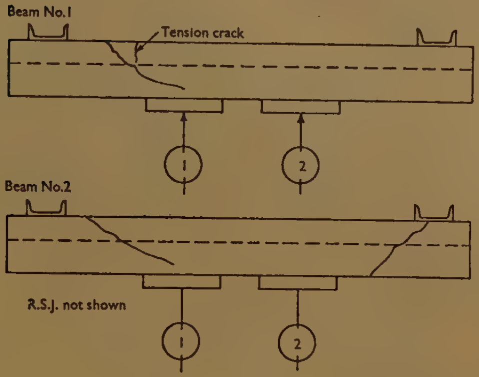
Fig. 44



Arrangement of Apparatus for Loading Test

TABLE 4

Beam Type 3 (Two beams tested)



n-Situ Topping

Tested at age of 14 days.
Mix : 1 : 2 : 4 unvibrated.

Pre-stressed Soffit

15″ × 3¾″ with castellations.
15″ wide × ½″ deep.
Sixty 10-gauge wires.
Area of steel = 0·772 square inch.

Shear Resistance

Permissible shear stress = 100 lb. per square inch.
Area taking shear = 15(5·51–0·50) square inches.

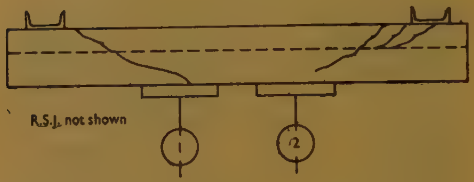
$$\text{Shear stress} = \frac{\text{shear force}}{15 \times 5{\cdot}01}$$

Test Results

	Load : tons		Shear stress : lb./sq. in.	Remarks
	1	2		
Beam No. 1	10	10	298	Appearance of tension crack No. 1 in pre stressed soffit at about 20″ from end of beam, and diagonal tension crack No. 2. See *Figs 34* and *35*. No further load on jacks possible.
	0	0	0	On unloading, tension crack No. 1 remained open.
Beam No. 2	16	16	477	Diagonal tension crack No. 3 appeared throughout depth of beam at approx. 45°.

TABLE 5

Beam Type 4 (Tested at age of 14 days)



In-Situ Topping

Mix : 1 : 2 : 4 unvibrated.

Pre-cast Soffit

15″ × 3¾″ with castellations.
15″ wide × ½″ deep.
Reinforced with sixteen ⅝″-diameter bars.

Shear Resistance

Permissible shear stress = 100 lb. per square inch.
Area taking shear = 15(5·52 − 0·50) square inches.

$$\text{Shear stress} = \frac{\text{shear force}}{15 \times 5{\cdot}02}$$

Test Results

Load : tons		Shear stress : lb./sq. in.	Remarks
1	2		
12	12	356	Slight diagonal tension crack appeared in soffit.
14	14	415	Diagonal tension crack extended over whole depth of beam at approx. 45°. See *Fig. 36.* No sign of horizontal cracks.

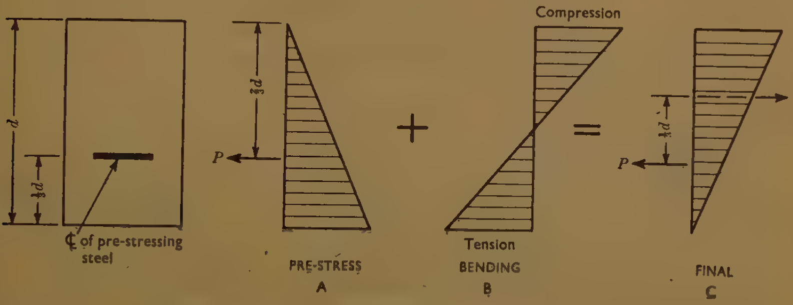
APPENDIX II

COSTS OF STEEL FOR VARIOUS ARRANGEMENTS OF PRE-STRESSED BEAMS

In construction A * the pre-stressed cables must be concentrated with their centre of gravity at the third point from the bottom of the beam. This produces the greatest compressive stress at the bottom, and no tension stress at the top. In *Figs 37* the additional-stress diagram (due to loading) is shown and also the final combined diagram, which in the ideal case, brings the stress at the bottom to zero, and at the top gives the same compressive stress that is produced by the load.

It can be seen that the centre of compression of the final diagram is at the upper third and, since all the tension is vested in the steel, the centre of tension must be at the lower third. In other words, the arm between the centre of tension and the centre of compression is one-third. This does not affect the compressive stress but it affects the quantity of steel required for the tension. Since the arm in normal reinforced concrete is 0·84 of the effective depth, that is, between 0·75 and 0·8 of the full depth, much of the economy of pre-stressing is lost. In Table 1 this is shown under the following assumptions :

Figs 37



TYPICAL RECTANGULAR PRE-STRESSED BEAM

$$\alpha = \frac{\text{permissible stress for high-tensile wire}}{\text{permissible stress for mild steel}}$$

$$\beta = \frac{\text{cost of high-tensile steel including pre-stressing}}{\text{cost of mild steel.}}$$

One method used to increase the economy has been the attempt to use I-sections (construction B). In this case, it is possible to concentrate the steel at a much lower point and still have zero stress at the top. In fact, with a good choice of section, the centre of gravity can be at the quarter-point from the bottom. With a symmetrical section, the centre of compression in the final diagram (see *Fig. 38*) will then be at the quarter-point from the top and the lever arm will be one-half, which is an improvement but still considerably less than with ordinary reinforced concrete.

Other methods of overcoming this difficulty have been to introduce reinforcement at the top of the unit (construction C) so that a tensile stress can be allowed at the top in the stressing diagram. (See *Figs 39*.)

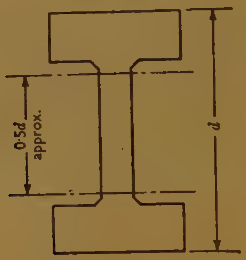
In this case, the reinforcement can be arranged nearer the bottom and possibly at the corresponding place to ordinary reinforced concrete. However, the centre of gravity of the final compressive stress is not affected by this, and although additional mild steel is needed, the lever arm will not exceed 0·57 of the depth in the case of the rectangular section or 0·65 of the depth in the case of the I-section.

---

* See Table 1, p. 232.

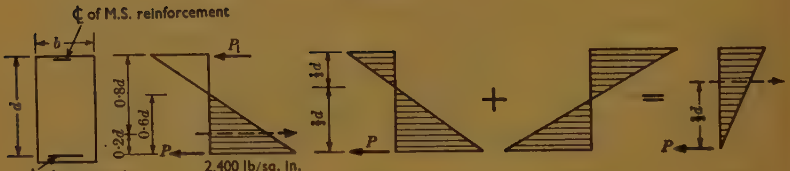
A method which is more successful is that of using ordinary reinforced concrete to carry the dead weight and to post-tension the beam before the live load is applied. In this case, before the post-tensioning commences, there is compression at the top and tension at the bottom, and it is possible, therefore, to have a stress diagram due to the post-tensioning with tension at the top and compression at the bottom. The tensile steel can then be arranged near the bottom surface. The optimum result will be arrived at if the dead and live loads are approximately equal, but it means that, for practical purposes, the pre-stressing is applied for half the safe load only, so that any advantages of pre-stressing are halved. Further, this procedure is practicable only with post-tensioning and calls for a very heavy dead load, usually found only in bridge construction.

Fig. 38



SUITABLE I-SECTION

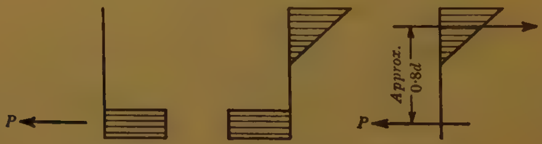
Figs 39



(a) PRE-STRESS (b) BENDING (c) FINAL (d)

RECTANGULAR PRE-STRESSED BEAM WITH MILD-STEEL REINFORCEMENT AT TOP

Figs 40



PRE-STRESS (a) BENDING (b) FINAL (c)

PRE-STRESSED SOFFIT (CALCULATED AS ORDINARY REINFORCED CONCRETE)

It may be seen from Table 1 that the costs of reinforcement are very much reduced in the two cases D and E, referring to composite construction. In both cases the centre of the steel is very near to the optimum point and, in addition, because the final-stress diagram does not extend over the whole section (see *Figs 40*), the centre of compression is approximately at the same point as it would be in reinforced concrete. In other words, the lever arm between the compression and tension is back to where it would be in normal reinforced concrete.

APPENDIX III

STANDARD CALCULATIONS FOR COMPOSITE T-BEAMS USING THREE STANDARD PRE-STRESSED SOFFITS AND IN-SITU CONCRETE TOPPING

Type	Section	Pre-stressing force in soffit after allowing for all losses
A	$22\frac{1}{4}'' \times 3\frac{3}{4}''$	150,000 lb.
B	$15\frac{1}{4}'' \times 3\frac{3}{4}''$	100,000 lb.
C	$8'' \times 3\frac{1}{4}''$	35,700 lb.

Allowable design stress in the pre-stressing material, after all losses have been accounted for, to be $\frac{1}{2{\cdot}25}$ of the ultimate breaking stress or $\frac{1}{1{\cdot}6}$ of the 0·1-per-cent proof stress, whichever is the smaller.

For example, using the Hoyer system of pre-stressing with 10-gauge wires :

0·1 per cent proof stress = 92 tons per square inch.

$\frac{1}{1{\cdot}6} \times 92 = 57{\cdot}5$ tons per square inch.

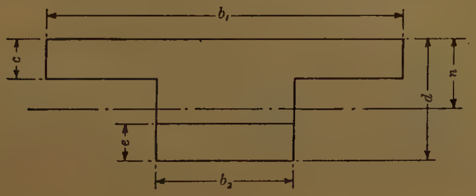
Ultimate breaking stress = 124 tons per square inch.

$\frac{1}{2{\cdot}25} \times 124 = 55{\cdot}0$ tons per square inch.

Therefore design stress = 55 tons per square inch.
Initial pre-stress = 70 tons per square inch.

For soffit type A, ninety-five wires were used.
($95 \times 0{\cdot}0129 \times 55 = 67{\cdot}2$ tons = 150,000 lb.)

When calculating the stresses in the beam due to dead and live loads, the use of a modulus of elasticity for the in-situ concrete of 1,500,000 lb. per square inch for the dead load, and 6,000,000 lb. per square inch for live load together with 6,000,000 lb. per square inch for the pre-stressed soffit, has been found to result in a deviation of about 5 per cent in the value of the stresses, compared with those calculated using a constant modulus of elasticity throughout. In the following calculations a constant modulus of elasticity has been used. When calculating dead-load deflexion, a value of 1,500,000 lb. per square inch must be used for the in-situ concrete.



	Moment of Resistance of steel	Depth from underside of unit to centre-line of steel
Type A .	$150{,}000\ (d–2'' - c/2)$	$2''$
Type B .	$100{,}000\ (d–2'' - c/2)$	$2''$
Type C .	$35{,}700\ (d–1\tfrac{3}{4}'' - c/2)$	$1\tfrac{3}{4}''$

Depth to Neutral Axis

Neglecting concrete below slab:

$$n = \frac{\tfrac{1}{2}b_1c^2 + b_2e(d - \tfrac{1}{2}e)}{b_1c + b_2e}$$

$$= \frac{\tfrac{1}{2}Rc^2 + e(d - \tfrac{1}{2}e)}{Rc + e} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where $R = \frac{b_1}{b_2}$.

Moment of Inertia

$$I = \frac{c^3b_1}{12} + \frac{e^3b_2}{12} + b_1c\left[d - \tfrac{1}{2}(e + c)\right]\left[n - \frac{c}{2}\right] \quad . \quad . \quad . \quad . \quad (2)$$

Case I.—Type A *Units as* T*-beams with* $2\tfrac{1}{2}$*-inch slab*

Width of flange $= b_1 = 22\tfrac{1}{4}'' + 12 \times 2\tfrac{1}{2}'' = 52\tfrac{1}{4}''$

Substituting for $R = \frac{52\tfrac{1}{4}}{22\tfrac{1}{4}} \doteq 2{\cdot}35$, equation (1) gives:

$$n = \frac{(2{\cdot}35 \times 2\tfrac{1}{2}^2 \times \tfrac{1}{2}) + 3\tfrac{3}{4}(d - 3\tfrac{3}{4} \times \tfrac{1}{2})}{(2{\cdot}35 \times 2\tfrac{1}{2}) + 3\tfrac{3}{4}}$$

$$= 0{\cdot}031 + 0{\cdot}386d.$$

Neglecting 0·031, $n = 0{\cdot}386d.$

Equation (2) gives:

$$I = (2\tfrac{1}{2}^3 \times 52\tfrac{1}{4} \times \tfrac{1}{12}) + (3\tfrac{3}{4}^3 \times 22\tfrac{1}{4} \times \tfrac{1}{12}) + 52\tfrac{1}{4} \times 2\tfrac{1}{2}(d - 3{\cdot}125)(0{\cdot}386d - 1{\cdot}25)$$

$$= 50{\cdot}4d^2 - 320d + 675{\cdot}8$$

$$Z_c = \frac{50{\cdot}4d^2 - 320d + 675{\cdot}8}{0{\cdot}386d}\ \text{in.}^3$$

Assuming the steel to be fully stressed, when the beam is loaded, the M. of R. of the steel equals 150,000 ($d - 2'' - 1\tfrac{1}{4}''$) and the extreme fibre stress in the concrete is given by:

$$f_c = \frac{150{,}000(d - 3\tfrac{1}{4}) \times 0{\cdot}386d}{50{\cdot}4d^2 - 320d + 675{\cdot}8}\ \text{lb. per square inch} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

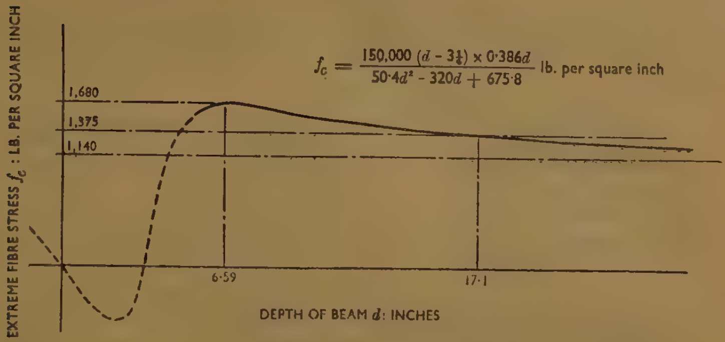
It can be seen that in a beam with the steel fully stressed, the depth of the beam determines the extreme fibre stress in the concrete, and their relationship is shown graphically in *Fig. 41.*

The maximum value of the extreme fibre stress in the concrete, found by differentiating equation (3) with respect to d and equating the result to zero, is 1,680 lb. per square inch with a depth of 6·59 inches.

On increasing the depth from 6·59 inches the extreme fibre stress in the concrete decreases from 1,680 lb. per square inch to 1,140 lb. per square inch with a very large depth as can be seen in *Fig. 41.*

Using a mix of 1 : 1½ : 3 (vibrated) for the in-situ concrete, the maximum permissible stress in the concrete (in accordance with the Code of Practice CP 114, 1948), is 1,375 lb. per square inch. In a beam with the steel fully stressed, the depths of the beam at

Fig. 41



which the extreme fibre stress in the concrete is 1,375 lb. per square inch are found from equation (3) :

$$1{,}375 = \frac{150{,}000(d - 3\frac{1}{4}) \times 0{\cdot}386d}{50{\cdot}4d^2 - 320d + 675{\cdot}8}$$

Therefore d equals 17·1 inches or 4·7 inches, the latter value being neglected because it is less than the minimum practical depth of the beam.

Therefore, on loading a beam of depth up to 17·1 inches, a maximum permissible stress of 1,375 lb. per square inch occurs in the concrete before the steel is fully stressed. The limiting stress is the concrete stress, and the M. of R. of the beam equals M. of R. of the concrete, which equals:

$$1{,}375\,\frac{(50{\cdot}4d^2 - 320d + 675{\cdot}8)}{0{\cdot}386d}\ \text{lb.-inches.}$$

On loading a beam of depth greater than 17·1 inches, the limiting stress is the steel stress, the maximum concrete stress being less than 1,375 lb. per square inch, and the M. of R. of the beam equals M. of R. of the steel, which equals 150,000 $(d - 3\frac{1}{4})$ lb.-inches.

For an economic use of steel, the steel should be fully stressed and to achieve this for a beam of depth up to 17·1 inches it is suggested that either the thickness of the slab be increased or a concrete used with a permissible stress of about 1,680 lb. per square inch.

Similar results for other soffits are given in the following summary.

SUMMARY

Type A *Unit :* $2\frac{1}{4}''$ slab T-beam.

$d < 17{\cdot}1''$ M. of R. $= \frac{1{,}375(50{\cdot}4d^2 - 320d + 675{\cdot}8)}{0{\cdot}386d}$ lb.-inches

$d > 17{\cdot}1''$ M. of R. $= 150{,}000(d - 3{\cdot}25)$ lb.-inches.

Twin Type A *Unit :* $4''$ slab T-beam.

$d < 11{\cdot}2''$ M. of R. $= \frac{1{,}375(116{\cdot}5d^2 - 900d + 2{,}288)}{0{\cdot}79 + 0{\cdot}313d.}$ lb.-inches

$d > 11{\cdot}2''$ M. of R. $= 300{,}000(d - 4{\cdot}0)$ lb.-inches.

Type B *Unit :* $2\frac{1}{4}''$ slab T-beam.

$d < 8{\cdot}38$ M. of R. $= \frac{1{,}375(38{\cdot}1d^2 - 239d + 498)}{0{\cdot}337d + 0{\cdot}197}$ lb.-inches

$d > 8{\cdot}38$ M. of R. $= 100{,}000(d - 3{\cdot}25)$ lb.-inches.

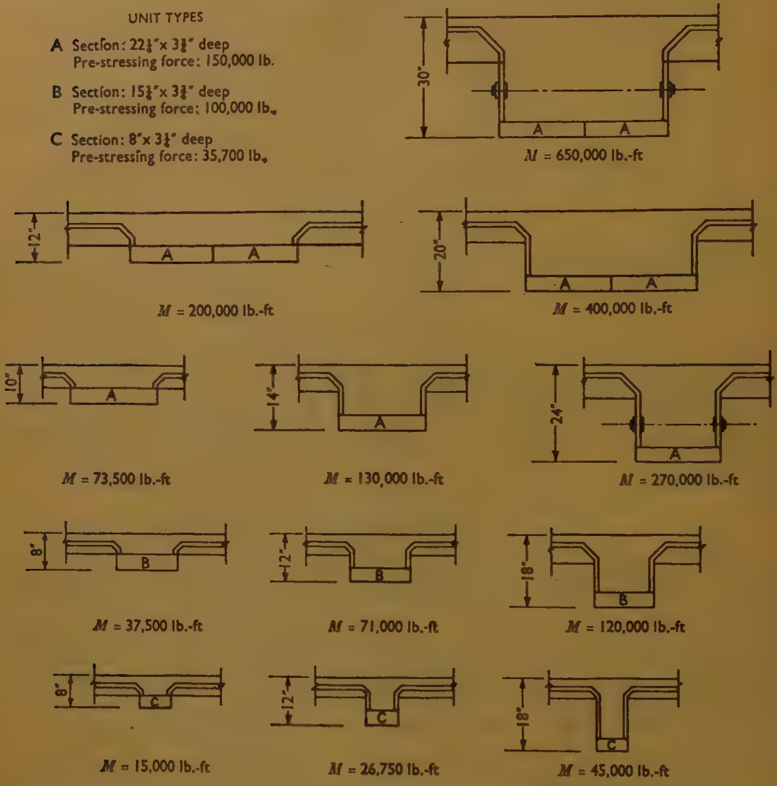
Type C *Unit :* $2\frac{1}{2}''$ slab T-beam.

d any value M. of R. $= 35{,}700(d - 3{\cdot}0)$ lb.-inches.

Applications:

The same unit can be used with different depths of beam for various bending moments. *Figs 42* show standard A, B, and C units together with cross-sections of beams for various bending moments ranging from 15,000 to 650,000 lb.-feet.

Figs 42



STANDARD "A," "B," AND "C" UNITS, TOGETHER WITH CROSS-SECTIONS OF BEAMS FOR VARIOUS BENDING MOMENTS

APPENDIX IV

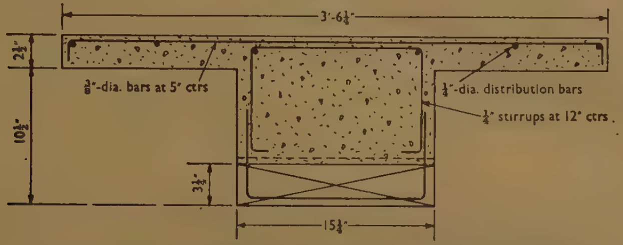
REPORT ON A FULL-SCALE LOADING TEST ON A COMPOSITE BEAM, CARRIED OUT BY R. HARRY STANGER ON 23RD NOVEMBER, 1951

The test was designed to determine the ultimate strength in bending of a composite T-beam made up of pre-stressed soffit and in-situ concrete topping.

The composite beam, supplied by The Liverpool Artificial Stone Co., was 23 feet 4½ inches long, and of the section shown in *Fig. 43*. The pre-stressed soffit was 15¼ inches wide and 3¾ inches deep, not including castellations, which were ½ inch high and extended over the whole width of the beam. Reinforcement consisted of sixty wires, 10-gauge, with a guaranteed ultimate breaking stress of 118 tons per square inch and a 0·1-per-cent proof stress of 92 tons per square inch. (Designed stress in steel $= \frac{118}{2{\cdot}25} = 52{\cdot}5$ tons per square inch.)

The in-situ topping was of 1·1 : ½ : 3 mix, vibrated. The beam was tested at an age of 18 days, and the compressive strength of the in-situ topping at that time was 6,270 lb. per square inch.

Fig. 43



Section through Composite Beam for Loading Test

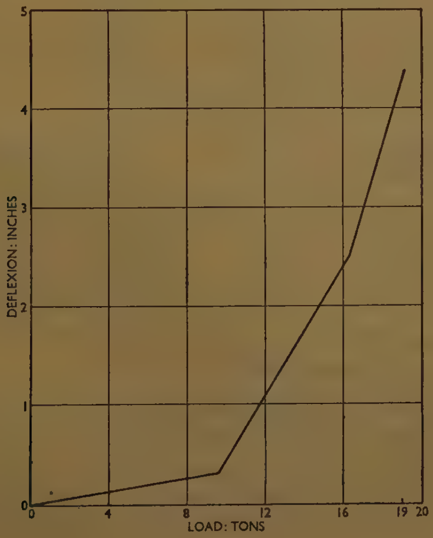
In the test, the beam was simply supported over a span of 22 feet 4½ inches and loads were applied at the fifth points by two jacks, the forces on the jacks being measured by means of proving rings. The arrangement of the testing apparatus is shown in *Fig. 44* (facing p. 249). Measurements of the central deflexion were taken (see *Fig. 45*).

The beam was first loaded up to 8·85 tons on the jacks, which was additional to a uniformly distributed dead load of 2·73 tons, equivalent to 1·14 times the design load. No cracks were seen and the deflexion was 0·29 inch. On increasing the load to about 10 tons, tension cracks appeared in the pre-stressed soffit in the centre of the span.

With a load of 12·4 tons, equivalent to 1·50 times the design load, on the jacks, the deflexion had increased to 1·196 inch and on removal of this load, the residual deflexion was 0·036 inch, showing a 97-per-cent recovery.

The beam was then loaded to 16 tons (1·88 times the design load), and the stress in the wires calculated to be 98 tons per square inch (that is, 0·15-per-cent proof stress, or 1·065 times 0·1-per-cent proof stress), and the deflexion to be 2·5 inches. The tension

Fig. 45



cracks in the soffit (in the middle fifth) were seen to be more numerous and they extended from the bottom of the beam to the underside of the slab. Upon removal of the 16 tons a residual deflexion of 0·11 inch showed a 95·6-per-cent recovery.

The beam was then loaded to 18 tons and showed a deflexion of 3·75 inches, which remained constant until the load was increased. Upon further loading, a small number of wires were heard to break. Ultimate failure took place, with an applied load of 19 tons, owing to the wires failing at a crack almost exactly in the centre of the beam. The crack, even at failure, did not extend into the compression slab.

The deflexion at failure was approximately 4½ inches, and the calculated stress in the wires 114 tons per square inch.

Throughout the test there was no sign of any diagonal tension cracks at the supports or horizontal cracks between the pre-stressed soffit or in-situ topping.

CONCLUSIONS

1. The test results are in accordance with the theory on which the calculations shown in Appendix III are based. The average stress of 114 tons per square inch in the wires at failure being 3·5 per cent less than the ultimate stress.

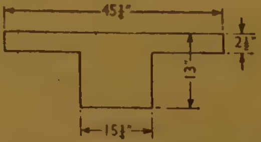
2. The overall factor of safety is 2·17, which compares favourably with normal reinforced concrete. It can be concluded that the use of a design stress of $\frac{1}{2 \cdot 25}$ of the ultimate breaking stress is justifiable.

3. On consideration of the deflexion up to the point of cracking of the soffit, an average value of Young's Modulus for concrete was calculated to be $5 \cdot 5 \times 10^6$ lb. per square inch.

4. The most important result is that it was confirmed that steel having no actual yield is superior to steel that yields, even beyond the actual pre-stressing when the beam acts like a normal reinforced-concrete beam. Present-day factors of safety on the compressive stress in reinforced concrete are sufficiently large to cover, with this type of beam, the increase in compressive stress due to the raising of the neutral axis when the strain in the steel is increased, so long as no actual yield takes place.

LOADING TEST—RESULTS

Using theory outlined in Appendix III



Length of beam = 23′ 4½″
span = 22′ 4½″

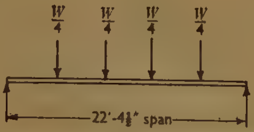
Before cracking of soffit
$I = 3{,}800$ in.^{4}
$Z_{top} = 830$ in.^{3}

Dead weight of beam
= 2·73 tons = 6,120 lb.

After cracking of soffit
$\left.\begin{array}{l} I = 1{,}005 \text{ in.}^4 \\ Z_{top} = 503 \text{ in.}^3 \end{array}\right\}$approx.

Pre-stressed soffit
15¼″ × 3¾″ with castellations 15¼″ wide × ½″ deep.
Reinforcement: sixty 10-gauge wires.
Area = 0·774 square inch.
Ultimate breaking stress = 118 tons per square inch.
0·1-per-cent proof stress = 92 tons per square inch.
Design stress $= \frac{118}{2 \cdot 25} = 52 \cdot 25$ tons per square inch.
Moment of resistance $= 52 \cdot 5 \times 0 \cdot 774 \times 2{,}240\ (13 - 3 \cdot 25)$ lb.-inches.
$= 883{,}000$ lb.-inches.

Loading at fifth points



Max. B.M. due to dead load = 206,000 lb.-inches.

„ B.M. due to loading $= \left(\frac{3}{20}\,.Wl\right) = 90{,}800\ W_T$

Where W_T = total load on beam in tons.

Deflexion due to applied loads up to point of cracking of soffit $= 0{\cdot}01572\,\frac{Wl^3}{EI}$

For $W = 4{\cdot}4$ tons, $l = 22'\ 4\frac{1}{2}''$, and $\alpha = 0{\cdot}141''$
$E = 5{,}500{,}000$ lb. per square inch.

TABLE 6

Load : tons	Deflexion : inches	B.M. : lb.-in.	Calculated stress in wires : tons/sq. in.	Approx. comp. in concrete : lb./sq. in.	Remarks
0	0	206,000	—	242	Initial conditions.
4·4	0·141	606,000	—	732	
7·45	—	883,000	62·5	1,065	Design load.
8·85	0·294	1,010,000	—	1,220	
10·4	0·584	1,152,000	—	2,290	Nine tension cracks appeared in soffit in centre of beam at 1·8 × design load.
12·4	1·196	1,332,000	84·0	2,650	
0	0·036	206,000	—	242	97% recovery.
12·0	1·140	1,296,000	—	2,580	
16·0	2·500	1,660,000	98·0	3,300	0·15% proof stress. No. of cracks increased.
0	0·110	206,000	—	242	95·6% recovery.
16·0	2·625	1,660,000	—	3,300	
18·0	3·750	1,842,000	—	3,670	Deflexion remained constant at this load.
19·0	4·5	1,932,000	114·0	3,850	Some wires were heard to break before ultimate failure took place.

Discussion

Mr John Cuerel agreed with the Author that in present-day circumstances, social conditions and so on, the battle between pre-cast concrete and in-situ concrete was tending to go in favour of the former—so much so that Mr Cuerel suggested that the Author was overstating the disadvantages of pre-cast concrete. Generally speaking, factories making pre-cast concrete components were located at the source of the aggregate, and in many cases had their own gravel pits or quarries. Moreover, the factories were usually quite close to the source of cement, so that there was not very much in the point that transport was a factor adverse to the use of pre-cast concrete.

The Author had also mentioned that there was a loss of general monolithic effect with pre-cast sections. Mr Cuerel questioned whether that was in fact quite so bad as the Paper might seem to imply. He had himself used pre-cast pre-stressed sections, for the deck of a jetty, in the form of planks 15 feet long and 4½ inches thick, six of which were placed side by side to form a panel 14 feet 3 inches square clear; they had not been covered with in-situ concrete, but merely grouted together. A concentrated test load of 16 tons had been put on the centre panel, and there had been a deflexion of about ⅜ inch. Although there had been a considerable overload, the grouted joints remained fully effective in connecting the planks, just tending to open at full load, but closing again on removal.

Although he felt that the Author had overstated the disadvantages of pre-cast concrete, he agreed that in many circumstances the use of pre-cast and in-situ concrete together had very great advantages over either used exclusively. The Author had referred to adhesion and to the principal stresses. The question of the shear and principal stresses being equal and the principal stresses being at 45 degrees was based on the use of a simplified theory of bending, which pre-supposed that the member was of great length in proportion to its depth and that the point which was being considered was remote from the loads or supports. In actual fact, beams were of considerable depth in relation to their span and often loads were near the supports. Mr Cuerel felt that at the junctions in the beams which the Author described, there was actually a vertical compression pushing the two parts together.

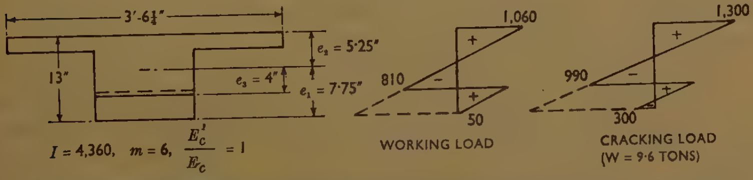
The tests which the Author had described were most interesting, but probably the addition of a little nominal steel connecting the pre-cast section with the in-situ section would be of great benefit. The suggestion seemed to be made that the concrete took the whole of the shear up to 100 lb. per square inch, and that when that reached 101 lb. per square inch it should all be taken by the steel. Mr Cuerel thought, however, that the one could be graded into the other, and that the nominal steel he had suggested would be of great benefit in preventing incipient or progressive failure at the junction. That should be, of course, in conjunction with good bonding and, in such conditions, castellations might not be of much importance.

Certain pre-cast inverted T-sections were shown in the Paper which he thought might well be made more robust. He had used them on bridge beams for spans of 60 feet or longer, and by tapering off the top of the web one could get the wires to be concentric at the ends. That being so, a pre-cast section which was more robust to start with could take the whole shear, and that was of great advantage in reducing the need for scrupulous keying of the in-situ to the pre-cast work.

Dr Paul Abeles congratulated the Author on the pioneer work which he had carried out in introducing the type of composite construction described. He agreed that that type should be more economical than both ordinary reinforced concrete and pre-cast pre-stressed concrete. However, he disagreed with one very important point; that was the approach which the Author had adopted towards the design of such composite members by considering them as ordinary reinforced concrete, the pre-stressed soffit being the reinforcement.

There were three types of members composed of pre-stressed and in-situ concrete. The first type was a pre-stressed reinforcement embedded in the in-situ concrete as, for example, at the top of continuous beams over the support shown by the Author. In that case Dr Abeles agreed that it had to be considered as reinforcement, since it formed only a small proportion of the tensile surface. In the second type, however, the pre-stressed pre-cast component comprised a substantial part of the tensile zone (for example, the entire soffit of a simply-supported beam) and acted together with the in-situ concrete as a homogeneous section like uniform pre-cast and pre-stressed concrete. That could be seen from the test results where, after previous loading up to 95 per cent of the ultimate loading, hardly any permanent deflexion remained on removal of the load, which did not occur with ordinary reinforced concrete. That indicated the behaviour of the type and one could not, in Dr Abeles' view, simply ignore that property and, neglecting the in-situ concrete in the tensile zone, merely assume nominal stresses on the lines of ordinary reinforced concrete.

Figs 46



Actual Stress Distribution in the Test Beam described in Appendix IV

In the third type almost the entire section was pre-stressed and only a small part was added in situ.

Dr Abeles referred to tests carried out by British Railways, Eastern Region, described in reference 1, p. 243, which had proved the co-operation of the two parts of a composite beam until failure.

Figs 46 showed the test the Author had carried out, investigated for a homogeneous section. Dr Abeles had considered a higher I-value than the Author, since the entire sectional area was taken into account. The tensile stress thus computed for the lowest fibre of the in-situ concrete was more than 800 lb. per square inch at the working load, and approached 1,000 lb. per square inch at cracking, where the cracking load was assumed as 9·6 tons from *Fig. 45.*

Based on the wrong approach, the Author had decided to call the *E*-values infinite or negative, when the composite beam remained horizontal or was hogging; but in fact the *E*-value must have remained the same in all cases of equal strength and stress. Either the shrinkage in the added concrete was greater than the creep in the pre-stressed concrete, and there was sagging, or it would be the other way round and hogging would occur even if the pre-stressed soffit was uniformly supported. In the event of the influence resulting from both shortenings being equal, the beam would remain straight.

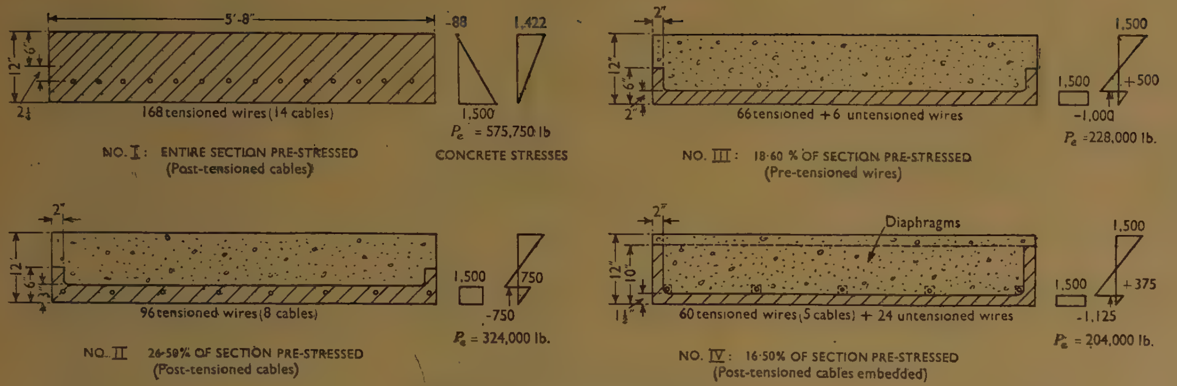
It was thus seen that such a composite beam was a partially pre-stressed construction with considerable tensile stresses under working load, which were, however, entirely harmless, as test results had shown. Any visible cracking would commence from the bottom fibre, where there were either compressive stresses or very low tensile stresses under working load. However, the Author had been very fortunate in having obtained, without difficulty, the approval of the authorities for his designs. He had achieved that by simply ignoring those appreciable though harmless tensile stresses, that was to say, by an incorrect approach.

It was possible to apply the advantages of composite members also to pre-cast construction and to make them in two stages, the first part being pre-stressed. That solution had been suggested first by British Railways, Eastern Region. *Figs 47* showed a comparison of four solutions for underbridges. No. I was the usual type of a fully pre-stressed slab, requiring 14 cables. The three alternative solutions II–IV represented pre-cast composite solutions in which no tensile stresses occurred at the bottom fibre under working load, whilst the respective bending stresses in the added concrete were 750, 1,000, and 1,125 lb. per square inch. The first stage, comprising a channel, contained in solutions II and IV post-tensioned cables (eight and five respectively), and in solution III pre-tensioned wires. The carrying capacities of all four cases were the same, and untensioned wires had to be added in cases II and IV. By that arrangement it was possible to halve the steel quantity, as could be seen from the Table.

Mr G. P. Manning remarked that there was a composite jetty being constructed in the south of Eire which raised one or two interesting points. A diagrammatic section of it was shown in *Figs. 48.* The slabs sat on pairs of cylinders spaced 24 feet apart, the bents being at 35-foot centres along the jetty. The cylinders were of composite construction, comprising pre-cast tubular sections one on top of the other and filled with in-situ mass concrete.

It had been decided to make the deck composite by casting the main beams in situ and pre-casting the secondaries. There were two methods of doing that: one could cast the main beams in situ first and leave pockets

Figs 47



NO. I: ENTIRE SECTION PRE-STRESSED
(Post-tensioned cables)

NO. III: 18·60 % OF SECTION PRE-STRESSED
(Pre-tensioned wires)

NO. II 26·50% OF SECTION PRE-STRESSED
(Post-tensioned cables)

NO. IV: 16·50% OF SECTION PRE-STRESSED
(Post-tensioned cables embedded)

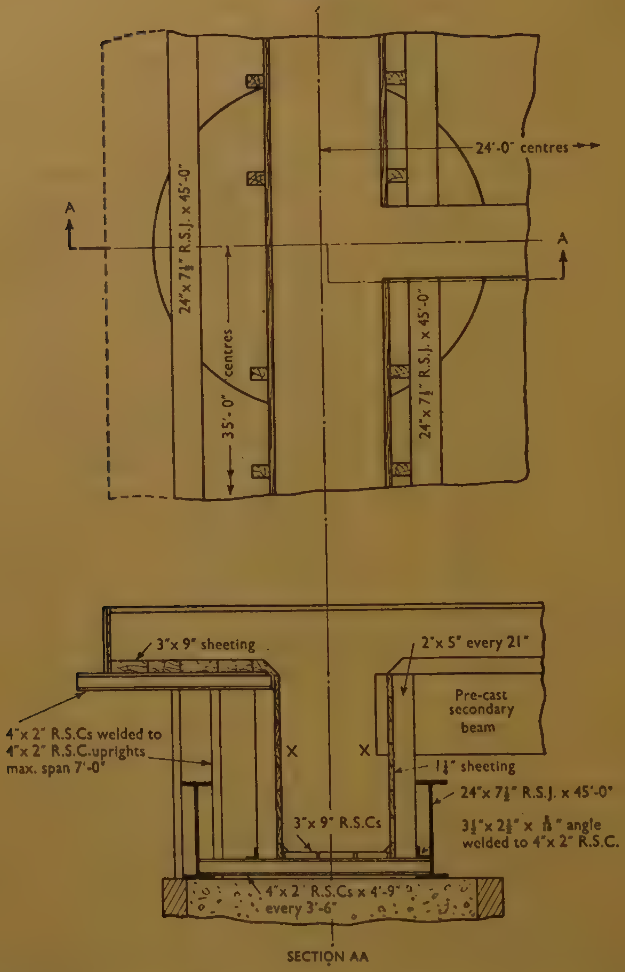
Type	P_e	No. of wires
	in per cent	
I	100	100
II	56	57
III	39·5	43
IV	33·5	50

All wires 0·20" in dia. (tensile strength, 100 tons/sq. in.)

COMPARISON OF PRE-STRESSED SLABS
M_w = 2,448,000 lb. inches, $Z = 68\,\frac{12^2}{6} = 1{,}632$ cubic inches, $f = \pm\frac{M}{Z} = \pm 1{,}500$ lb. per square inch)

FOUR ALTERNATIVE SOLUTIONS OF PRE-STRESSED CONCRETE SLABS FOR RAILWAY UNDERBRIDGES

Figs 48



Junction of Secondary Beam with Main Beam for Composite Jetty

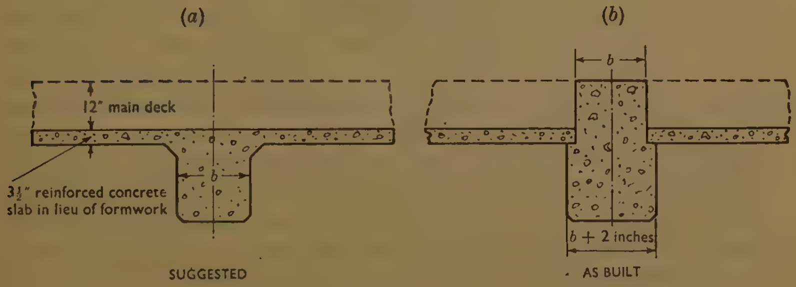
in them, and then place the pre-cast secondaries in those pockets afterwards, or one could place the pre-cast secondaries on the falsework with the ends projecting, and then cast the main beam in situ afterwards. The latter made a sounder job with simpler details, but the falsework would carry both the weight of the pre-cast secondaries and the weight of the main beam, whereas in the former method there was more difficult detail, with a lopsided load, but the falsework carried only the weight of the main beam.

The next point was the shuttering for the deck. In order to avoid timber shuttering it had been decided to use thin pre-cast slabs for shuttering, and it had originally been suggested that they should be cast as part of the secondary beam, as shown in *Fig. 49* (*a*); but the contractor had adopted the usual method of simply enlarging the width of the secondary and putting the thin slabs across, spanning from secondary to secondary, which were at 8-foot–9-inch centres (see *Fig. 49* (*b*)).

Another interesting point was that the shuttering for these pre-cast secondaries was neither timber nor steel, but consisted merely of two solid blocks of concrete, with a device for pushing them slightly apart to strike it.

The question of the joint at X (see *Figs 48*) raised a point which sometimes arose with this type of construction, which was that the load from the unit above might cause extremely heavy tension locally across the section XX, and that was sometimes very difficult to deal with.

Figs 49



THIN PRE-CAST SLABS USED AS SHUTTERING

He thought that the Author was unduly worried about that problem of composite construction, which in fact arose very often, because in Great Britain it was the standard practice to cast the rib of a main beam first and then cast the slab on top. In good practice the rib was cast one day and the slab the next, but in the event of delay occurring (caused by, say, a break-down of machinery, a strike, or the intervention of a weekend), there might be an interval of 2 or 3 days or even weeks, so that the occurrence was more common than one would think.

Dr K. Hajnal-Kónyi disagreed on various points in the theoretical approach described in the Paper. First, on p. 236 the Author had said "The composite construction varies considerably from the usual method using pre-stressed concrete, when the whole of the beam is pre-stressed. With the latter method, the neutral axis moves upwards from the bottom edge if the load is increased beyond the design load, with the result that the steel stress is reduced and the concrete stress is increased as compared with the calculated stresses assuming the neutral axis to remain at the bottom edge of the beam." Dr Hajnal-Kónyi could not understand the alleged difference between composite construction and the "usual method of using pre-stressed concrete when the whole of the beam is pre-stressed." The actual behaviour of both types of construction, when loaded to failure, was exactly the same. If some designers calculated stresses on the basis of the assumption that the neutral axis remained "at the bottom edge of the beam," that proved only that they had a wrong approach to the problem and not that the two types of structure behaved in a different manner.

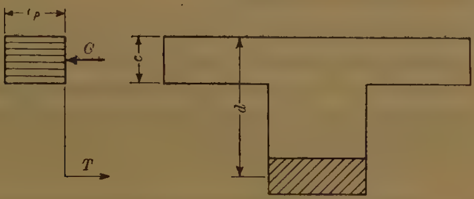
The Author went on to say: "This provides a hidden factor of safety (so far as the steel is concerned), which is not there if only the tension zone is pre-stressed. On the other hand, with the usual pre-stressed construction, the factor of safety on the compression is reduced, but it remains the same when only the tension zone is pre-stressed, until the stress in the steel exceeds substantially the limit of proportionality." How would the Author define the factor of safety? Dr Hajnal-Kónyi thought that the factor of safety was the ratio between ultimate load and design load, and he did not know what was meant by the "factor of safety on the compression." A beam could be designed to fail either in tension or in compression. Failure in compression was undesirable, and most designers tried to avoid it. From that point of view he did not think that it was fair to say, as the Author had said, that, in the more usual type of pre-stressed concrete, failure would occur first in the concrete. Most designers attempted to ensure that failure would take place in the steel. It was entirely in the designer's hands to provide more or less steel or more or less concrete, in order to anticipate failure in tension or in compression.

The practical application was in the Author's method of design. In Appendix III, designs had been developed for three types (A, B, and C), and an equation was given for the extreme fibre stress in the concrete. The Author had concluded from that equation "that in a beam with the steel fully stressed the depth of the beam determines the extreme fibre stress in the concrete." *Figs 41*, illustrating type A, showed that the concrete stress as calculated by the Author's formula was, in fact, very little affected by the depth of the beam. In order to reduce the concrete stress from 1,680 lb. per square inch to 1,375 lb. per square inch (a reduction of 18 per cent) it was necessary to increase the depth of the beam from 6·59 to 17 inches (by 160 per cent)!

Dr Hajnal-Kónyi was of the opinion that the Author's formula was not a suitable basis of design, since it did not reflect the actual behaviour of the beam if loaded to failure. The stress f_c obtained from that formula did not give a measure of the safety of the beam. What mattered was whether the force C which could be developed by the compression flange was greater or smaller than the force T which could be developed by the reinforcement. That could be assessed in a simple manner as numerous experiments had proved.

Figs 50 showed a section of a beam with a pre-stressed pre-cast member. It was safe to assume that C was equal to 0·6 × cube strength × area of the flange. A further assumption which could be made was that c was equal to or less than one-half of the total effective depth d. If those two conditions were fulfilled, namely, that the area multiplied by 60 per cent of the cube strength was equal to or greater than the tensile force which could be developed by the wires, and that the depth c of the compression flange was not greater than half the effective depth of the beam, then the member would fail in tension. With that type of construction it was very easy to design it in that manner. When that rule was applied, smaller limiting values were obtained than those given on p. 256. He did not know what concrete strength the Author had assumed, but his straight-line distribution obviously necessitated a greater effective depth than the rule just enunciated would call for. The test in *Fig. 43* was carried out with a concrete of 6,270 lb. per square inch cube strength, so that the decking was very much under-reinforced, and half the cube strength would have been sufficient to cause failure in tension.

Figs 50



Mr F. Walley, referring to the bonding of the pre-cast to the in-situ work, agreed that it was dangerous to rely on the rough surface between the two units, but said that in some tests using pre-cast slabs on top of pre-cast units, a very much higher failing load had been obtained than by simply using a timber decking on top of those units, so that a composite action of the two pre-cast units acting together was obtained without in-situ work.

He exhibited three slides illustrating the effect of castellations, the first showing secondary beams with castellations, the second and third showing the composite action of those under test. The second slide showed the ordinary pre-stressed unit with an in-situ slab cast on top, and the considerable deflexion when approaching failure; no crack had appeared between the in-situ and pre-cast concrete. The third slide showed the beam at failure, again with no cracking between the in-situ and pre-cast concrete.

He thought that the Author, in the compilation of Table 1, had been a little unkind to other forms of pre-stressed construction, because in the calculations no note had been taken of the possibility in pre-stressed construction of designing only for the live loads and not for the combination of dead and live loads, nor had account been taken of the device of two-stage pre-stressing to cut down the stresses and so reduce the section. Mr Walley had compared the area of steel used in the example given in the

Paper with those shown on his slides; he had obtained practically the same area of steel—a difference of only 2 or 3 per cent—but the area of concrete was only about two-thirds of that shown in the Author's design, so that at least there would be a considerable saving in the concrete.

Mr Walley thought that the Author, in his method of design, had sacrificed the homogeneity of the material and could not use the whole cross-sectional area to resist the applied load, and so could not make full use of the possibilities of the high-grade concrete which was available. He could not use the stresses of 500–600 lb. in tension during erection, which Mr Walley had used quite successfully.

Mr Walley could not agree with the statement on p. 235 that a wide shallow beam in pre-stressed concrete was often more economical than a deeper one. He found that extremely difficult to believe, and would challenge it if it referred to pre-stressed concrete construction. It would be interesting to know whether any further proofs of that could be put forward.

On p. 236 the Author had discussed the results of the test which he had carried out, and had stated that " the first hair cracks appeared in the unit when, according to calculation, the pre-stress was a little more than offset by the tension in the bottom due to load." Mr Walley had been through those calculations and had not got the same answer. The Author's result was a stress of about 800 lb. per square inch, but if the whole section was taken into account, as Dr Abeles had suggested, a value of about 300 lb. per square inch effective tensile stress was obtained.

The Author's method of design could have been much simpler if it had been based on ultimate-load theory, as Dr Hajnal-Kónyi had said. Having done that calculation, his test result agreed with the result of a method which Mr Walley had outlined in a discussion on previous Papers before the Institution, in which a failing load of a little more than 2,000,000 lb. had been recorded, whereas the Author had obtained a little more than 1,900,000 lb., taking the full ultimate strength of the wire into consideration, and not his figure of 114 tons per square inch. If the Author had calculated using ultimate-load theory, he would have obtained better agreement.

It was not clear whether the proposed factors on p. 237 were based on the test results, as seemed to be implied, because in one of the conclusions of the test it was stated that the figures were justified by the tests and not the other way round. The Author was a very brave man if he drew conclusions from one test, and in any case the figure of $\frac{1}{2{\cdot}25}$ bore no relation to the safety factor of the structure, because if a proof stress had been used for initial tension in the wire, the ultimate strength of the member would have been the same. The practical limit of the steel stress was probably the creep of the steel.

On pp. 237 and 238 reference was made to the modulus. It was misleading, Mr Walley thought, to use that; the Author should really use the product EI, but he did not think that it was possible to state the value of I for a composite section with a variable modulus. It was true that beams would hog, as the Author had stated—if they were pre-stressed and did not carry any live load on them—because of creep and shrinkage in the bottom slab. In Appendix III, the Author had stated that, using variable moduli for his materials, he got a difference of only 5 per cent. Mr Walley found that difficult to believe, because if one used the two moduli to which the Author referred, one being a quarter of the other, the effective width of the top flange would be only a quarter of the bottom width, so that the I would be quite different, and the stresses would have a difference of more than 5 per cent. It was not clear why, since the Author had considered it as a reinforced concrete design, he had not based it on ordinary reinforced concrete design, in which case the depth would not have been to the neutral axis, but to the centre.

In the same Appendix the Author had given an expression for the maximum value of concrete stress. That seemed to be quite unnecessary. It did not seem to have anything to do with the argument, and must be quite wrong, because if the depth were reduced or increased the concrete stress came down, and if it were reduced below 6 inches Mr Walley did not believe that the concrete strength would go down. He thought that the differentiation there was based on a wrong premise.

Mr R. D. Binns referred to a job at Willesden where the construction consisted of pre-cast columns (which were held in sockets left in the concrete base) supporting rectangular-section pre-stressed concrete beams, which in turn supported a composite pre-cast concrete rib and hollow pot floor (see *Fig. 51*). Further details of this job were shown in *Fig. 52*, in which the castellations in the top of the pre-stressed beams could be clearly seen. The pre-cast floor ribs had a top steel bar, as mentioned in the Paper, to which stirrups protruding from the ribs were welded, so that during construction the flooring ribs acted as steel beams, supporting the wet weight of concrete.

For about 2 feet on either side of the pre-stressed beams, the hollow pots were omitted from the flooring, so that when the floor was concreted a composite T-beam was formed.

There was a similar application at London Airport, where beams of 40 feet span, with castellations and nominal protruding stirrups, had been combined with pre-cast flooring and in-situ concrete to form a composite T-beam.

There was one advantage in the use of composite pre-stressed concrete, not mentioned by the Author, which was particularly applicable to marine work. By pre-stressing the soffit of beams, the concrete is compressed where it is most needed, that is, where the beam is most subject to corrosive action.

Since some people appeared to be worried about the effect of fire on composite pre-stressed concrete construction, he pointed out that a particular composite pre-stressed concrete rib and hollow pot floor had withstood a 2-hour fire test satisfactorily, with no indication that fire had any more detrimental effect on the composite floor than it would have had on an ordinary monolithic floor.

There seemed to be little mention in the Paper of the question of propping. The Author had shown beams 3¼ inches deep which were spanning about 20 feet during construction, and which would obviously have to be propped at close centres if they were to carry the weight of wet concrete. What method of propping did the Author adopt to prevent the pre-stressed soffit unit cracking during construction? If the props were low the soffit would crack, whilst if they were too high the top of the unit would crack.

There had been a report recently issued by a well-known Institution in which it was suggested that the in-situ concrete to be used in composite construction should be of the same grade and quality as that for the pre-stressed concrete with which it was combined. Had the Author any view on this matter?

Mr A. L. Benjamin stated that a considerable amount of research had been carried out in Sweden on the question of castellations on the upper surfaces of pre-cast and pre-stressed beams, and tests had shown that roughened top surfaces were adequate to form a key between the pre-cast and in-situ concrete.

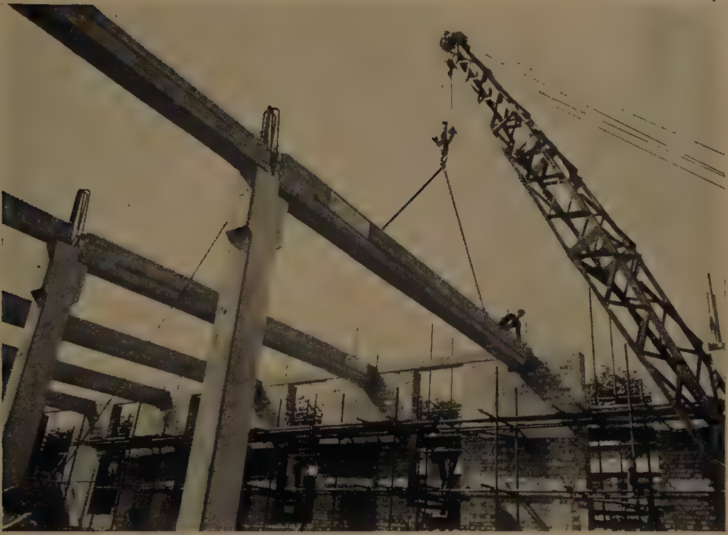
Pre-stressed beams were being produced and used in Sweden without castellations, relying solely on the roughened surfaces for the transfer of the stresses. There were also beams being produced and used with what he would call semi-castellations, being less well defined than those illustrated in the Paper. That form of castellation was most economically produced by forcing a shallow former into the concrete during manufacture, thus avoiding the necessity for special shuttering.

With regard to the use of pre-cast pre-stressed soffits of beams, as distinct from complete webs of T-beams, Mr Benjamin had found that the problems of transport, handling, and propping those slender soffits often outweighed the advantages gained in other directions. The deep web afforded a simply-produced member which was easy to transport and erect.

With regard to Table 1 of the Paper, it should be borne in mind that the Table referred only to steel costs and did not take into account the relative propping and shuttering costs. The comparison with a normal reinforced-concrete beam seemed to assume the same permissible working concrete stresses in a normal beam as in a pre-stressed beam—a condition which did not apply in current practice.

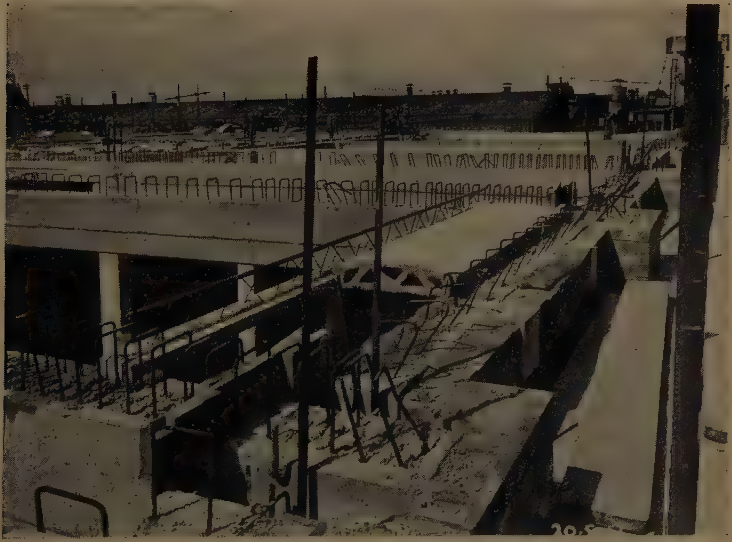
Professor L. T. Oehler explained the principle on which he had been working with regard to composite construction at the University of Texas. There had been a good deal of work in the U.S.A. on "joistile" construction in connexion with ordinary reinforced concrete, and he had been hoping that pre-stressed concrete could be used in the same way. *Figs 53* showed a cross-section of a joistile test beam which was pre-stressed by tightening

Fig. 51



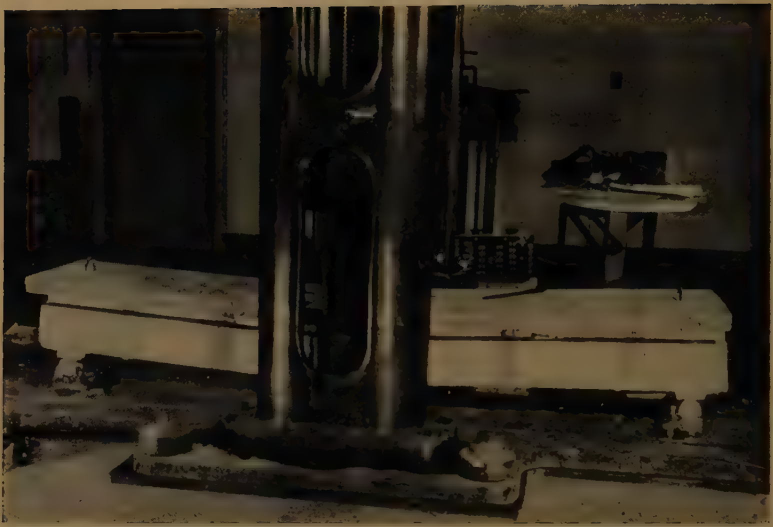
Pre-cast Columns supporting Rectangular Pre-stressed Beams for Floor shown below

Fig. 52



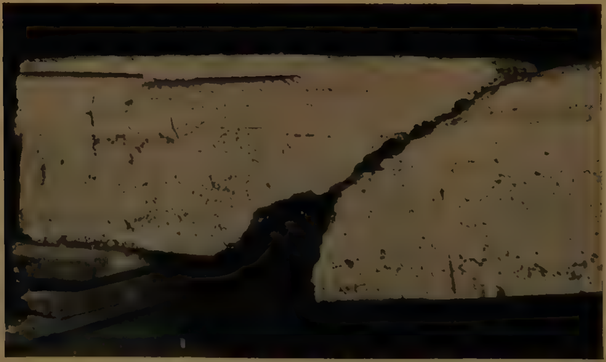
Rectangular-Section Pre-stressed Concrete Beams supporting Composite Pre-cast Concrete Rib-and-Hollow-Pot Floor

Fig. 54



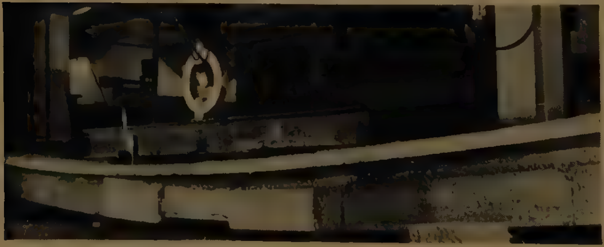
Test Beam after Failure, showing the Large Crack which opened between the Third and Fourth Tiles after Bond Failure had occurred

Fig. 56



A Failure by Excessive Diagonal Tension

Fig. 57

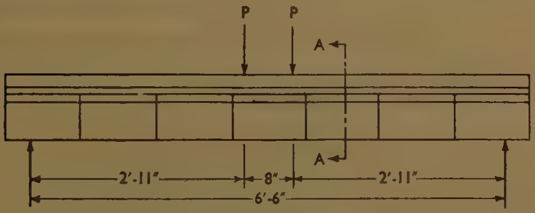


A 14-foot Composite T-beam under Test

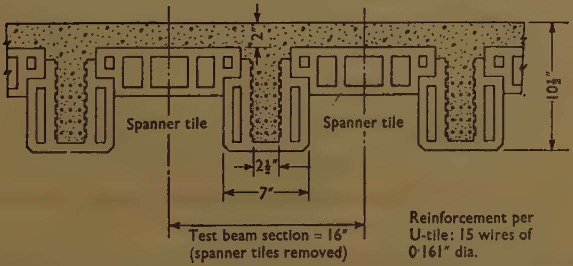
the wires against end anchorage blocks bearing on the ends of the tile. After the tile was pre-compressed the concrete was poured—forming in effect a T-section as shown in *Figs 53.* The pre-stressed wires were cut at the end of the beam, after the concrete had hardened and bonded to the wires and the tile, thus forming a composite section where the tile alone was pre-compressed and the concrete poured in-situ without pre-compression.

To test the feasibility of that type of construction a beam had been formed and tested. Joints between the structural clay tile had been "buttered" and the wires had been pre-stressed to 120,000 lb. per square inch. Twenty-six days later, when the concrete had been poured, the stress in the wires had dropped to 112,000 lb. per square inch. At the end of 10 days the wire stress had dropped to 107,000 lb. per square inch, and at that time the wires had been cut away from the end anchorage blocks which were bearing on the tile. That operation had not affected the stress in the wires, except locally at the ends of the beam. The electrical strain gauges attached to the wires had been made waterproof, which had ensured their reliability throughout the entire period. They had thus provided continuous stress readings previous to testing as well as during the actual test. The beam had been tested 12 days after pouring the concrete, and

Figs 53



ELEVATION OF TEST BEAM

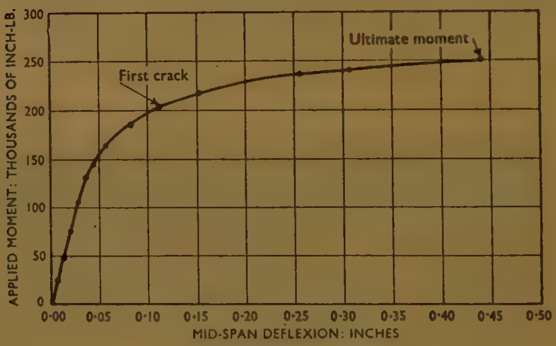


SECTION AA
(CROSS-SECTION OF TYPICAL FLOOR SLAB)

"JOISTILE" TEST BEAM

whilst the results were not conclusive they did indicate that that type of construction was feasible, and with its inherent advantages might definitely prove worth-while. The beam had failed at a total live load of 14,300 lb., or with an applied moment of 250,000 inch-lb., owing to progressive bond failure starting with the bottom wires at a moment of 223,000 inch-lb. and proceeding successively to the wires above. At the time each wire failed in bond the average stress in the wire at mid-span had been 200,000 lb. per square inch and the average bond stress over the embedded length of the wire had been 208 lb. per square inch. *Fig. 54* showed the test beam after failure had taken place, with the wide crack which had opened between the third and fourth tiles because of the bond failure of the wires. The deflexion of the beam at mid-span with increase in applied moment was shown in *Fig. 55*. The elastic deflexion had been somewhat greater than calculations based on the full transformed area of the composite section would indicate. The first noticeable crack had appeared in the mortar between the tiles at an applied moment of 202,000 inch-lb., which corresponded to a tension stress of 750 lb. per square inch at the extreme fibre, calculated on the basis of the full transformed area of the section. At no time during the test had there been any indication of slip between the tile and the concrete, even though there had been considerable differential stress between the two, owing to the fact that only the tile had been pre-compressed.

Fig. 55



VARIATION OF MID-SPAN DEFLEXION WITH APPLIED MOMENT

The following merits of that type of construction warranted more extensive research :—

1. Commercial tiles might be obtained which had compressive strengths of 8,000 lb. per square inch—sufficient for storing large pre-compression forces.

2. With that method, end anchorage blocks were not permanently attached—allowing the repeated use of that costly item.
3. The pre-stressing force was applied directly to the tile without the need for auxiliary anchorages or stretching beds, as in conventional pre-stressed bonded construction.
4. With the large number of sizes and shapes of tile available, economical floor systems for nearly any span or loading were possible.
5. In actual practice, the stem portion of the effective cross-section could be pre-cast on the ground, lifted in position, the spanner tile placed, and the top portion of the concrete poured without the use of shuttering.

Mr Adolf Goldstein observed that a good deal had been said about the analytical and theoretical side of the design of the various units, but would the Author give some idea of the limits of their application ? It would appear that the Author had made out a case for them, and clearly there were applications, but Mr Goldstein wondered whether they would be so effective economically and practically in, say, longer spans, or under more normal conditions. He was referring there to conditions which, unfortunately, were not very prevalent in Great Britain, but there were normally instances where the straight-forward job with minimum site handling could be more economical than the more complicated methods, using two or more stages of construction, which the Author had described.

Would the Author say if the pre-tensioned bars which had been used as reinforcement in the case of continuous construction were economical and perhaps generally applicable. The job which had been illustrated certainly looked a " man-sized " job, but he wondered how the Author would apply those bars in a job where there were continuous beams in two directions. The bars were of considerable section, and Mr Goldstein was apprehensive about the problems of design and detail if it were necessary to use them in three planes.

Notwithstanding the fact that the bars themselves were pre-stressed, under load they would elongate and cause cracking in the concrete. Could the Author comment on that ? It would certainly be comforting to the designer to know that, under working loads, the concrete over the support did not crack.

He had not realized that the Karachi hangar edge beams had been pre-stressed before the shell had been cast. He wondered whether that was in fact an economic proposition, because if the beam was pre-stressed as soon as it was constructed, when the shell was cast on top of the beams the complete benefit of the pre-stressing action on the shell would be lost. In long spans that could be quite important, because the transverse bending moment in the shell was very much reduced by pre-stressing the beam and shell as a unit. By pre-stressing the beam first, formwork to the beam soffit could be removed more quickly, which was a favourable practical consideration. On the other hand, there was still the necessity for formwork to shell soffits in casting the shell, and since in practice contractors frequently liked to brace their propping to the beam soffits and bird-cages for the shell soffits together, the saving in shuttering might not be as great as would at first be thought.

Professor A. L. L. Baker said that he would like to say a few words in support of the Author's work. The advantages of composite construction were quite obvious, but the important point was whether or not the joint between the pre-cast work and the cast in-situ work satisfactorily resisted the horizontal shear. Two speakers had referred to tests, one in Sweden and one, presumably, a Ministry of Works test, and he would like to add his support to the results referred to. A number of composite beams had been tested at Imperial College, and in each case failure was caused by bending, and there were no signs of failure in the horizontal joint, which was made with a roughened surface without any castellations.

Mr E. F. Humphries said that since he had at one time had the privilege of association with the manufacture of some of the units described in the Paper, he would like to point out that the castellations were *not* easily formed. In terms of man-hours, it took at that time—possibly production methods had been much improved since those days—approximately three times as long to put the castellations on the top of the unit as it took to concrete it. Unless the procedure of concreting was slowed down very much, the casting of the main body of the units proceeded at a rapid rate and the placing of the castellations on top, which had to be done afterwards, was very much slower, with the result that the castellation became a separate block on the top of the main concrete. The Polish labour which had assisted in that operation had sought to help by putting 2-inch wire nails to tie the blocks down to the main concrete, but the Author had treated those units by going round with a hammer and chisel and knocking each castellation. If it had stood one blow it was all right; if it had not, and the castellation had come off, the unit had been rejected.

Mr Humphries had visited a factory in Switzerland where Stahlton floors were being made, and he had been much impressed by that type of construction, where an indented wire was laid through grooved clay tiles. Although the Author had said that concrete was used, at the Swiss factory a grout only had been employed to bond the tiles to the indented wire. Very large quantities were being made in standard lengths, about 1 to 5 metres long, and when a local builder wanted to put up a block of flats he asked for so many of those units of various dimensions and from the standard lengths the exact lengths required were cut with a circular carborundum saw. Builders could thus draw on that factory and get the units which they required immediately from stock. It seemed a great pity that no manufacturer, so far as he was aware, had been able to bring the Stahlton floor to Great Britain, because he felt sure that it would be readily taken up.

⁂ **Mr Stephen Revesz** stated that he had some experimental data in connexion with the adhesion between pre-cast and in-situ concrete of composite T-beams, where no special provision had been made for horizontal shear. He had carried out a number of such tests,[17] mostly using a pre-tensioned web, where failure had resulted from horizontal shear in one case, at a calculated shear stress of 134 lb. per square inch. The surface of the pre-cast concrete unit along the joint had been smooth. Another test by Purandare,[18] on a normal reinforced-concrete composite beam, had resulted in horizontal shear failure at 141 lb. per square inch. Where the surface had been slightly roughened it was found that 253 lb. per square inch could be exceeded. The Author's suggestion of forming castellations, seemed to be an excellent solution for raising the horizontal shear resistance without the addition of stirrups.

Fig. 56 showed a composite beam[17] which had failed at a diagonal tension of 480 lb. per square inch. A large mass of concrete had broken off around the wires; the angle of rupture was approximately 40 degrees.

The Author's observation that hair cracks could not be detected in the flange of composite pre-stressed beams did not necessarily mean that they did not exist in that zone. In tests at City & Guilds College (see *Fig. 57*) the flange thickness was only 1½ inch for an overall depth of about 8 inches. Although cracks could not be found in the flange with a magnifying glass, strain readings on the concrete had shown the neutral axis to be about ¾ inch from the top of the section.

The reference in the Paper to the "hidden factor of safety" was perplexing. It was perhaps only a consequence of the Author's view, that in non-composite beams failure "will take place *first* in the concrete."

From Mr Revesz's experience, the following had been confirmed by test results:—

1. Failure due to fracture of steel was not a special characteristic of composite construction, and might occur in ordinary pre-tensioned beams.[17]
2. A composite beam might fail owing to the extensive elongation of the steel, followed by the rise of the neutral axis, causing the crushing of the concrete.[19, 20] Although the steel did not break, its stress at failure closely approached its ultimate strength in pre-tensioned beams, and the crushing of the concrete was only an apparent cause of failure, and may thus be regarded as a secondary effect.
3. Primary compression failure occurred in "over-reinforced" beams, in which the maximum compression force developed in the concrete was smaller than the maximum tensile strength that could have been developed by the reinforcement. That type of failure

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⁂ This contribution was submitted in writing.—SEC. I.C.E.

[17] References 17 to 20 are given on p. 276.

was distinct from (2) where due warning was given by large deflexion and wide cracks, whilst in case (3) failure was sudden in pre-tensioned beams.

Amongst the many factors that influenced the failure characteristics (in bending of pre-stressed beams) were :—the elongation of the steel at its ultimate strength ; the pre-stress ; the steel percentage ; the concrete strength ; and the condition of bond between the steel and concrete.

References

[17] S. Revesz, " The Behaviour of Pre-stressed Beams." Lond. Univ., M.Sc. Thesis.

[18] N. N. Purandare, " The Effect on the Strength of Tee Beams of Casting the Web and Slab Separately." Lond. Univ., M.Sc. Thesis, 1939.

[19] K. Hajnal-Kónyi, " Test of a Railway Bridge Girder." *Concr. & Constr. Engng*, vol. 35, p. 296.

[20] P. W. Abeles, " Further Notes on the Principles and Design of Pre-stressed Concrete." *Civ. Engng and Pub. Works Rev.*

The Author, in reply, first dealt with a general matter which would answer several speakers in the discussion.

There was a great danger that the issue might be confused, and it was desirable to avoid that at the outset. The Paper discussed a method for pre-stressed concrete and not the plastic theory nor the question of factors of safety. He agreed that the present Codes of Practice did not deal logically with all problems, but if he wanted to compare one method of construction with another, particularly if the second one had to be carried out in accordance with the Code of Practice, it was essential to assume that the Code of Practice applied to both. He had to assume certain permissible concrete stresses, particularly for in-situ concrete ; he had to assume a linear distribution of stresses and disregard plasticity ; he had to assume that, however unsatisfactory he might feel it to be, the factor of safety had nothing to do with the ultimate failure but merely constituted the ratio of the permitted stress to the ultimate stress.

If pre-stressed concrete was to have the benefit of a more modern outlook on those points, so also should ordinary reinforced concrete and other materials and that would not have any effect on their comparative values.

Turning from the general to the particular, Mr Cuerel had mentioned that the principal stresses were not necessarily inclined at 45 degrees in very short beams. The Author agreed but mentioned that he had found that under test, even beams with a relationship between length and depth of 6, showed cracks at 45 degrees if there was insufficient diagonal reinforcement. That would indicate that only with exceptionally deep beams would a special investigation be necessary.

The Author agreed also on the question of the method of calculation of shear stresses with regard to steel. Again he had followed the method acknowledged by the Code of Practice, but if an improved method could be evolved, that would be advantageous.

He was particularly interested in the information given by Dr Abeles, especially with reference to the test carried out by British Railways. The Author fully appreciated that a tensile stress in the concrete could be taken into account, as was done frequently with partially pre-stressed sections, but that would be outside present Codes.

Mr G. P. Manning had suggested that the Author was unduly worried about the question of shear. It was true that for in-situ concrete the upper part of a beam was often cast separately, but it would not be more than a day or two after the lower part, and he always made it a condition in the specification that the time lag should not be greater than that. With pre-cast concrete, however, the soffit might be cast months before the top decking, and the concrete of the soffit would have reached an entirely different state by then. Also, in an ordinary reinforced-concrete beam there was shear reinforcement which could be avoided in a composite construction (particularly pre-stressed) if the shear was taken by mechanical bonds, such as castellations.

Turning to the points raised by Mr Benjamin, Mr Revesz, and Professor A. L. L. Baker, the mere fact that the information given by Mr Benjamin was contrary to that of Mr Revesz showed that ordinary bond between two types of concrete was very unreliable in shear and that many different results could be obtained—depending on the type of surface, the age of the concrete, and many other circumstances. Remarkably enough, the results given by Mr Revesz confirmed almost exactly results that the Author himself obtained and which he had not thought were sufficiently representative to be mentioned. He had found that units without castellations and a comparatively smooth top surface failed at approximately 150 lb. per square inch. He still did not think that merely "rough" surfaces should be trusted to transmit shear.

Dr Hajnal-Kónyi had stated that the factor of safety with the standard method was not quite unknown because in actual fact the neutral axis might be assumed to move upwards before failure. That also was undoubtedly correct, but, with the same justification, Dr Hajnal-Kónyi could condemn all calculations which were carried out in accordance with the theory of elasticity. The fact still remained that the officially accepted methods of calculation did not, at present, allow for that type of calculation, and therefore, comparing the calculation with the test, the Author had mentioned an extra factor, although he might be wrong in calling it "unknown." However, unless the dead load were sufficient for the pre-stressing wires to be placed near the bottom edge of the beam, the effective arm was still reduced with the standard method, as compared with the composite method. As to the question of stresses in the concrete, that again was a matter that would be of less importance if the Codes of Practice could be ignored.

The calculation given to explain the limiting depth of a beam for which it was unnecessary to calculate the compressive stresses, was of course, not identical with the actual calculation of the unit. As an approximate first calculation the Author found it very useful to calculate pre-stressed concrete as though it was ordinary reinforced concrete with the pre-stressed unit taken as the reinforcement. A considerable number of beams had been tested, all of which bore out the above, and he could assure Mr Walley that he did not base the whole theory on only the one test which had been given as an illustration.

However, he agreed with Mr Walley that, with post-tensioned concrete, where the weight of the in-situ concrete was sufficient to offset any tensile stresses at the top of the beam caused by the pre-stressing wires being placed close to the bottom surface, it was possible to obtain better results than those shown in Table 1.

Regarding the economy, or otherwise, of shallow beams in pre-stressed concrete, the situation was quite simple. The total cost of reinforced concrete consisted of the cost of the steel, concrete, and shuttering or moulds. Both concrete and shuttering were cheaper with a shallow beam, but the steel was more expensive. In pre-stressed concrete the steel costs were considerably reduced, compared with ordinary reinforced concrete, and therefore the economic depth was smaller.

So far as the use of varying moduli was concerned, it was impossible to give a general statement as to how much difference the use of that method would make. The reference in Appendix III was for a T-section only, and for such a T-section, which had a wide compression flange, the effect of a variable modulus of elasticity was comparatively small.

Mr Binns had mentioned the question of propping. Since pre-stressed units were comparatively thin, temporary propping during erection was necessary, and the stresses arising from the dead weight of the unit had to be taken into consideration. The Author had never found difficulties arising from incorrect propping, and it had been his experience that most contractors were quite capable of placing the props correctly. He had read with surprise the report mentioned which suggested that the in-situ concrete used should be of the same grade and quality as that for the pre-stressed concrete. He could not see any reason why the two should be equal, nor was it usually practicable to make them so.

The Author was indebted to Professor Oehler for his very interesting remarks on joistile construction, and the tests being made in the U.S.A. Those joistiles were obviously very similar to one of the proprietary articles mentioned in the Paper..

Mr Goldstein had asked whether composite construction would be useful for long spans as well as short spans. The Author believed that they would, although he confessed that he did not know of any example carried out of more than 48 feet span. He could imagine that that method would prove suitable even for long bridges, and he supposed that in that case both the pre-stressed part and the top part would have to be cast in situ, although carried out in two separate processes.

With regard to continuous beams, there were a number of difficulties. It was quite easy to use that method for continuous beams provided that the depth available was not less than about 14 inches. With very shallow beams it was difficult to place the pre-stressed units at the top over the supports. Even greater problems arose where two beams crossed each other, that was to say, where both the main beams and the secondary beams were pre-stressed. He would not say that all those points had been solved.

Mr Humphries had stated that it was difficult to form castellations but in that connexion his experience seemed to differ with most other experience.

Correspondence on the foregoing Paper is now closed, and no further contributions, other than those already received at the Institution, may be accepted.—Sec. I.C.E.

PUBLIC HEALTH ENGINEERING DIVISION MEETING

11 March, 1952

HENRY FRANCIS CRONIN, C.B.E., M.C., B.Sc., Vice-President I.C.E.,
Chairman of the Division, in the Chair.

The following Lecture was delivered and, on the motion of the Chairman, the thanks of the Division were accorded to the Lecturers.

"The Impact of Radio-activity on Public Health Engineering Services"

by

Arthur Key, D.Sc., Ph.D., and Arthur William Kenny, M.A., B.Sc.

THE phenomenon of radio-activity was first recognized by Becquerel in 1896. For 50 years the subject was largely of academic interest only, the practical uses being confined to cancer therapy and luminous paints. There was much talk about the liberation of huge amounts of energy by atomic disintegration, but the general public remained sceptical—until 1945, when an atomic bomb was exploded over Japan. The possibilities of atomic energy were then effectively demonstrated and since that time a vast amount of work has been carried out on its exploitation. The result is a completely new industry which, in common with other industries, has its peculiar trade-effluent problems. We wish to speak about this to-night, but before doing so it is necessary to say something, however briefly, about radio-activity itself.

Some of the basic facts about it are by now well known, but since the latter part of the Lecture will be dependent upon them a brief repetition will not be out of place.

We ask to be permitted to simplify matters a good deal; we must of necessity omit references to phenomena which are either of minor importance, for the time being at least, or of no possible concern to us to-night. Remembering this, we may say that when an atom undergoes radio-active change it throws out either an alpha particle or a beta particle and, in most cases, a gamma ray also. These changes occur in the nucleus of an atom and, as a result, the electrical charge on the nucleus is changed and the atom is transmuted into another element. Thus radium gives off an alpha particle which has a positive charge of two units; the remaining nucleus has therefore two units less of positive charge and is now the nucleus of a radon atom. Active potassium gives off a beta particle with a negative charge of one unit, leaving the nucleus with a positive charge one unit greater than before, which is now the nucleus of calcium. The chemical properties of an atom are determined by the charge of the nucleus, but all the atoms of an element are not necessarily of the same weight. Thus, while all the nuclei of carbon atoms have a positive charge of 6 units, the most common form has a nucleus weight of 12, but there is another with a nucleus weight of 13 and still another with a nucleus weight of 14 and this is radio-active. These varieties of atoms of the same nuclear charge are called isotopes.

It seems important to mention that radio-activity, although so recently discovered, is no new thing; it has always been with us, and it is everywhere. The walls of this room are radio-active; so is the food we eat; so is the water we drink. The examples of active elements we have just quoted have been deliberately chosen as being amongst those which occur naturally. Radium, of course, is well known. It becomes transmuted to radon, which is a gas and which becomes diffused in the atmosphere over the world. Radon itself is radio-active and changes into a whole series of radio-active solids which are washed out of the atmosphere by rain so that even the purest mountain stream is not free from radio-activity. Some other natural waters contain much more activity, having become contaminated with it underground.

Further, all potassium contains a minute proportion of an isotope which, over millions of years, slowly but inexorably changes into an isotope of calcium, giving off a beta ray in the process. The potassium in our bodies and our foods is no exception to this.

A small quantity of radio-active carbon (C^{14}) is also continually being produced in the atmosphere by bombardment by extra-terrestrial particles and this, as carbon dioxide, enters through photosynthesis into all organic matter; it has been detected in sewage sludge gas.

What is new about the present situation is not radio-activity itself, but the facts that:

(*a*) it is now being produced artificially;
(*b*) it is associated with elements which are not naturally radio-active; and
(*c*) it is being produced in relatively enormous quantities.

Radio-activity can be produced artificially by bombarding the nucleus of an atom. To put it crudely, if you can hit an atomic nucleus you may either knock a bit off, add a bit on, or split it into two. In any of these events an atom of a new element is created and the probability is that it will be radio-active. All this, however, is easier said than done. An atom is exceedingly small and the nucleus occupies only a very small fraction of its total volume—it is not a very large target. A further difficulty is that the nucleus carries a positive electrical charge and therefore strongly repels any particle which is charged in like manner. On the other hand, if a negatively charged missile is used, it is repelled by the electrons surrounding the nucleus, for they are negatively charged too.

For many years the only missiles available for nuclear bombardment were the emissions from naturally occurring radio-active substances. Success was first obtained by Rutherford soon after the first world war, and during the following decade a number of transformations were brought about in laboratories, each following the general pattern, element plus radiation giving rise to a different element plus a different radiation.

All these bombardments were carried out with missiles which were charged electrically—no other suitable ones were known—and a large proportion failed to find their target because of the repulsion either by the nucleus or by the surrounding electrons. There would clearly be less chance of these repulsions being effective if the missile could be made to approach the atom at a very high speed. This is the idea behind the various accelerating machines (the cyclotron and the Van der Graaf accelerator) of which the first was invented by Professors Cockcroft and Walton, who have recently been awarded the Nobel Prize for the work. These inventions greatly increased the efficiency and scope of the nuclear reactions that were now becoming possible.

In 1932, Chadwick discovered a new radiation as a product of one of these nuclear reactions. He called it the neutron, because it carried no electrical charge. It is this discovery which made possible the phenomenal advances in nuclear science which have occurred in the past 20 years. Being uncharged, a bombarding neutron suffers neither repulsion nor deflexion on its way to a nuclear target. It is therefore a missile *par excellence* for effecting nuclear changes, and it was immediately applied in this way. Numbers of new isotopes were thus artificially produced, many of them radio-active, and a link between natural and artificially produced radio-active elements was forged when one of the former was produced in the laboratory by nuclear bombardment. It was established that there was no essential difference between the two; they both obey the same laws, which we shall briefly describe below.

The next big step forward was in 1939, when a new nuclear reaction was discovered. Previously they had been of the type we have described, namely, element plus radiation giving a different element plus a different radiation. The new process can be described as element plus neutron giving rise to two different elements plus several neutrons. In the first type of reaction the radiation enters the nucleus and a small part of the swollen nucleus is thrown out as the different radiation, leaving a different element whose nucleus is only slightly different in weight (mass) from the original one. The new type (which only occurs with very heavy elements) is the fission process, in which the nucleus, swollen with the entering neutron, breaks into two approximately equal parts and some neutrons are thrown out. In an almost literal sense the atom is split. The nucleus does not always split in the same place, so that when the reaction is operated in bulk a whole series of fission products are formed, and they are all radio-active.

Now the neutrons which are produced as a result of fission are themselves capable of initiating fresh fissions which produce more neutrons. Under suitable conditions, therefore, the reaction is self-sustaining and even self-propagating, a feature which is the basis alike of the atomic pile and the atomic bomb.

Atomic piles are a source of energy, plutonium, fission products, and neutrons. The energy is liberated almost entirely in the fission process, and it may eventually be economical to use it. It is being used for space heating at Harwell already. The plutonium is produced from uranium by a nuclear reaction of the first type, namely, uranium plus neutron giving plutonium plus gamma radiation. It can be used as explosive material for an atomic bomb. The neutrons can be used to irradiate inactive materials placed in the pile to produce, by means of nuclear reactions, radio-active materials of known type. These find many uses in medicine, industry, and research. The fission products themselves are generally useless and constitute an entirely new type of industrial waste.

The radio-active materials produced by irradiation in the pile may also become waste after use, and since they may be used in any part of the country they may become waste in any part of the country.

In all, then, as more and more atomic piles come into operation and as the potentialities of radio-active isotopes become more widely recognized we must expect increasing amounts of radio-active industrial waste. We think we shall meet the interests of most of our audience if we concentrate on the liquid effluents, for it is these which may reach and affect our sewers, sewage-treatment plants, rivers, and water-supplies—in short, public health engineering services.

The questions we must always ask regarding a liquid trade waste are :—What are its properties ? Is it dangerous in any way ? If so, are there any limits of concentration below which the danger vanishes ? Will it affect the processes of treatment of sewage or water ? Can the dangerous constituents, if any, be removed in any way ? After that we may be in a position to say how it should be dealt with. For many trade wastes it would be superfluous to ask some of these questions. With a new type of waste we are not in that fortunate position, and at the risk of lengthening the lecture unduly we must say something on all these points.

We have said that radio-activity is the disintegration of elements accompanied by the emission of one or more types of particle or ray. This is a process which proceeds at a rate which is entirely uninfluenced by the chemical or physical condition of the isotope. The rate is characteristic for each isotope and is proportional at any time to the amount of isotope present unchanged. This type of change is characterized by a constant half-life period ; that is to say that the time required for a given quantity of activity to diminish to one half is a constant. In double that time it will diminish to one quarter, and so on. For instance, the activity of any iodine 131 declines by half every 8 days—it has a half-life period of 8 days. One way of solving a problem of waste active iodine, therefore, is to store it for some time. In 80 days, for instance—10 half lives—the activity will be diminished to $(\frac{1}{2})^{10}$ or to less than 0·1 per cent of its original value. There will, no doubt, be a wide field of use for delay tanks for this purpose, but they would not readily be applicable to, say, radium, which has a half life of 1,590 years. Half lives vary from a fraction of a minute to thousands of million of years—that of uranium, for instance, is 4,500 million years.

Alpha particles are the nuclei of helium atoms expelled with a certain velocity. Beta particles are electrons, similarly expelled. Gamma rays are very short, or hard, X-rays. All these rays have energy, and the amount of energy varies with their source. An alpha particle from radium, for instance, possesses only 91 per cent of the energy of a similar particle from polonium.

The most important property of these emissions is that of producing ions when passing through matter. This means that they detach from an atom part of the electron cloud which surrounds the nucleus. The most heavily charged particles produce most ions and the greatest concentration of them, but the production of ions requires energy derived from the radiation, which is soon brought to a stop. Alpha particles therefore produce a high concentration of ions, but are not very penetrating, whereas gamma rays produce a smaller concentration, but have a high range of penetration. Since the danger to health is largely on account of this ion production it follows immediately that inside the body alpha rays are much more dangerous than gamma rays, but outside the body gamma rays are the more dangerous because alpha rays cannot penetrate the tissue, unless they originate within a few millimetres of it. Beta particles occupy an intermediate position.

We have mentioned danger to health. This is one of the main points we have to keep before us; but to avoid giving a wrong impression it may be worth while to digress for a moment to explain that radio-activity has its useful side, even in relation to health. Considerable quantities of active materials are now being used in certain hospitals, largely for cancer treatment. The use of radium in that connexion is fairly well known and this element may be replaced by artificially produced radio-active sources. In a different way, radio-phosphorus is used for leukaemia and polycythemia vera, radio-iodine for thyroid cancer and hyperthyroidism, and radio-gold for some other cancers. In industry and the academic world the uses of radio-activity range from tracing the diffusion of gases and liquids to the study of arsenical poisoning, from the discharge of static electricity to the measurement of the spin of atomic nuclei, from the investigation of frictional wear to the study of the action of pesticides.

Likewise to the public health engineer radio-activity may have its uses, though these may not be spectacular. Although experiments in the sterilization of food by radiation have gone a long way, the day when we can sterilize water by subjecting it to the effects of a radio-active substance is not yet in sight. There is, however, considerable scope for the use of radio-activity as a tracer. For instance, it may find application as a tracer for following the underground flow of water. For this and many other purposes, active materials have one great advantage over other substances in that they are detectable in extremely minute concentrations, so that the amount added will not change the specific gravity of the water and bring about movements which would not otherwise occur. The use of salt, for instance, always has this disadvantage. It is not an application which could be made lightly, however, for the radio-isotope would have to fulfil special requirements. It would, for instance, have to be harmless in any concentration at which it could be pumped from the ground. At the same time, the concentration would need to be sufficiently high to permit ready estimation. The half-life period would have to be sufficiently long to allow it to be followed perhaps for several weeks. And its chemical nature would have to be such that it could not enter into combination with any substance likely to be met with underground, nor enter into cation or anion exchange reactions. In the same way, and perhaps with less difficulty, radio-activity may be employed in investigating the dispersion of sewage in river or sea.

A similar use in detecting leaks in water mains has also been suggested, but conditions here are not favourable. The amount of radio-activity required to permit radiations being detected through a covering of soil would be far too large for safety and would present difficult handling problems.

The most obvious use of radio-activity at present, however, is as tracer material in research work. It may greatly assist in the design of reaction tanks and other parts of plant and in elucidating the mechanism of the chemical and biological reactions on which purification processes rely.

However, none of these or other uses should be undertaken without the advice of a specialist in this field and without giving full consideration to possible danger to health of the isotopes employed. To this subject we must now return.

The first recorded instance of damage to the body by radiation occurred 50 years ago. Becquerel, the discoverer of radio-activity, carried about some radio-active material in his pocket and found that his skin adjacent to the material became " burnt." Since that time there has been a very wide experience of the effect of radiations on the body.

Before the danger was appreciated, radium-mine workers were affected by it, and also workers with luminous paints. Radiologists were also affected by the X-rays with which they worked, and since these rays, too, cause damage through the ions they produce, knowledge of it is of assistance in assessing the danger from radio-active emission. In addition, there is now considerable experience in the use of radiation in attempts to cure disease, particularly cancer. Then we know a great deal about the radiation damage to the people of Nagasaki and Hiroshima. We have the experience of protecting a large number of people who have been working on atomic energy during the last 10 years, protection which is acknowledged, we believe, to be highly successful. Finally, there is a great deal of information as a result of special researches. We mention these points to indicate that in this matter of the effect of radio-activity on health the medical profession is not working completely in the dark, as might have been supposed with a new subject. All the radiations affect health because of their ionizing properties and the experience in one field bears on problems in another.

We cannot here give any account of the nature of the biological effects of radiation. It is sufficient to state that they can be serious, even fatal, at sufficiently high concentrations, but that at sufficiently low concentrations they are either non-existent, or readily repairable by the normal processes of the body, or negligibly small. The whole question of deciding what concentration is sufficiently low is not one for engineers or chemists, but for the medical profession with the advice of specialists in other fields.

There are three chief ways in which the body may receive radiation. It may receive it from its surroundings. An atomic pile, for instance, radiates neutrons and if there were no protection a body near it would be irradiated. This is readily prevented by interposing between the pile and the body sufficient matter (usually concrete) to absorb the neutrons so that the number reaching the body is below the permissible limit. It is possible, too, that sufficient active material may concentrate on the walls of a sewer to make it dangerous to work in it. On this, we can only say that such an accumulation cannot occur in this country at the present time, though we are afraid there will be no time for us to justify that view to-night.

Or a man may inhale air containing radio-active particles which may be retained in the lungs, perhaps indefinitely. Radio-active argon is formed in air used for cooling atomic piles and the purpose of the high chimney stacks at such installations is to disperse the activity in such a large volume of air that the concentration is harmless.

Or a man may ingest radio-activity in water or food, and the ingested activity may pass to any part of the body and stay there; it may, alternatively, be excreted within a short time. It is this method of receiving radiation we want to deal with in greater detail, for it arises when a liquid effluent is discharged to a stream which is later used as a source of water supply. Fish from the stream may also be used as food.

The damage done by radio-activity taken into the body depends upon a number of factors including :—

(*a*) The type and energy of the radiation. We have already explained how the quantity and concentration of ions are controlled by these factors.

(*b*) Whether or not the activity is retained in the body, and for how long. This is obvious; an element ceases to irradiate the body as soon as it leaves it.

(*c*) Whereabouts in the body it concentrates. Different organs may be sensitive to different extents. Different organs are of different size; it is obvious that a given activity concentrating in the thyroid, which weighs 20 grammes, will be more concentrated than if it were dispersed throughout the muscle system, which weighs 30,000 grammes. Other things being equal it would do much more damage.

With adequate information on all these points it is possible to calculate what the activity in an organ would be if the concentration of active element in, say, drinking water were maintained at a given value for a life-time. By fixing a very safe value for the former, an equally safe value for the latter can be obtained. Into such calculations a safety factor is always introduced to cover individual idiosyncracies, for instance, the person who habitually drinks twice as much water as the average and the person whose organs might be particularly sensitive to radiations.

We must now pause to explain what we may term the "principle of pessimistic assumptions." We said earlier that a safe concentration of the activity due to any element in water can be calculated, provided information on certain points is adequate. Sometimes this is not available; and the principle is that in cases of doubt the worst should be assumed. For instance, if we do not know for certain whether an element is retained in the body or excreted from it then it must be assumed to be retained, for that would be the more dangerous case. If we have a mixture of active elements of different degrees of danger and we do not know how much of each is present then it is assumed that all the activity is due to the most dangerous of the elements. If we do not know whether an active element passes out with sewage effluent or is retained in the sludge then we assume it all passes out with the effluent when we wish to know whether that is safe, but we assume it all stays in the sludge when we want to check that that is safe. We shall give one or two examples of this later.

Before giving values for the life-time permissible concentrations of certain radio-isotopes in drinking water, we must define the unit in which they are expressed. This is the "curie," a disintegration rate of $3{\cdot}7 \times 10^{10}$ atoms per second. This is the number of disintegrations which occur in a gramme of radium, and originally the definition referred to the equivalent of activity to that present in one gramme of radium. It should be noted that the unit refers to a rate of disintegration and not to an amount of material. Thus, while one curie of radium is contained in one gramme, one curie of iodine 131 is present in no more than 0·000008 gramme. The reason is that the half-life period of iodine is only a few days, compared with 1,590 years of radium, so that a much greater proportion of the number of atoms present at any time will disintegrate in the next second, even though in each case all the atoms are radio-active and will disintegrate eventually.

There are about eight hundred radio-active isotopes of the ninety-eight known elements and it is possible to work out the life-time safe concentration of each of them in water, incorporating appropriate safety factors (or factors of pessimism, as we may call them, in accordance with what we said earlier) where information is doubtful and further similar factors to cover individual idiosyncracies, and so on. Not all these calculations have, in fact, been made. Some of them probably never will be, since the isotopes concerned have half-life periods of minutes or less so that there is no real possibility of their ever getting into drinking water. In other cases values for life-time tolerance concentrations have been suggested by a number of individual scientists, but these are not accepted by our Ministry. It is felt, and, we submit, rightly felt, that where the safety of water-supplies is concerned, the views of no single scientist, no matter how distinguished, can be accepted as being sufficiently authoritative. Indeed, the distinguished scientist would probably share our view, for he would realize that the decisions would involve the pooling of the knowledge of experts in a variety of fields. We look therefore to the Medical Research Council for authoritative figures, and their Protection Committee has given long consideration to the matter. The following life-time drinking-water tolerance concentrations have been approved by them :—

Alpha emitters	
Radium	4×10^{-10} micro-curies per cc.
Other alpha emitters	$2{\cdot}4 \times 10^{-9}$,, ,, ,,
Beta emitters	
Radio-calcium and radio-strontium	2×10^{-8} ,, ,, ,,
Other beta emitters	10^{-6} ,, ,, ,,

To be strictly accurate, what has been laid down is not a general drinking-water tolerance concentration, but a concentration which it is safe to add to the River Thames, in addition to its natural activity, bearing in mind that it is the source of water-supply for eight or nine million people and making no allowance for the possible decay of activity during storage and forgetting the possibility (even likelihood) that some of the activity may be removed during filtration. It seems safe to assume that general drinking-water tolerances, when they are required, will be of similar magnitude.

There are now four things that must be said, though very briefly, about these figures. The first is that although a life time has been taken as about 70 years there will be no danger to people who live longer. We have spoken of factors of safety and would now emphasize that they are very large. We do not suppose that the Medical Research Council would immediately revise their figures even if it were suddenly discovered that the average expectation of life had doubled or trebled.

The second point is that the laying-down of a permissible concentration does not mean that it must at no time be exceeded. The danger to be safeguarded against is of accumulation of activity in the body over a long period, and so long as on the long-term average the prescribed values are not exceeded all will be well. In practice we insist that during any period of a month the activity should not exceed a total calculated from the figures quoted. Within that period irregularities may and do occur, though we like (and obtain with important discharges) a reasonable degree of uniformity.

Thirdly, the figures are mutually exclusive; that is to say that if the added radium is equivalent to the concentration quoted, there must be no other activity discharged. It may be argued that in some cases this may not be necessary, but in other cases it clearly is, so that the safest thing to do is to require it in all cases.

Fourthly, the figures are extraordinarily small. In the case of radium the tolerance concentration is 4×10^{-10} parts per million; in the case of radio-calcium it is $1{\cdot}2 \times 10^{-12}$ parts per million. It is plain therefore that no purely chemical analysis could possibly be used to determine the concentration of active elements. This brings us immediately to the question of activity determination, for it is clearly no use laying down permissible concentrations unless it can be checked that they are being observed.

We cannot here describe those instruments which are used in activity determinations. It is sufficient to say that they can detect and register alpha or beta particles or gamma rays, as the case may be, which reach them and that by their use the number of such emissions arising from a suitable, and suitably placed, sample can be calculated.

A suitable sample must contain sufficient activity to be measured and it must not contain too much inactive material which would prevent the emitted particles from entering the counter. This requirement immediately presents us with a difficulty when determining the activity of waters, because although the activity may be concentrated by evaporation a considerable amount of inactive solid matter concentrates with it, particularly in the case of hard waters. With these the activity due to, say, strontium at life-time tolerance level is not far above the limit at which measurement becomes very uncertain.

Further, the various counters available count the disintegrations, but they only give the merest hint, if that, of what the activity is due to. They do not tell us whether it is from a dangerous element or not. To get any further we require a radiochemical analysis.

This is easier said than done. Although radiochemical separation is essentially chemistry, it is not to be lightly undertaken even by a qualified chemist. Special techniques are required and the whole process is time consuming and expensive. Even then any sort of complete separation is almost impossible, and certainly impossible on a day-to-day basis.

We are thus left in the position of having to ensure that rivers used as a source of drinking water must not contain activity beyond certain quite minute concentrations, which vary from element to element, and being without any practicable method of determining directly whether a water conforms to the requirements or not.

Before explaining how this situation is being met, may we digress and say something on the questions, " What happens to radio-activity during purification processes to which sewage and water are subjected? Are those processes affected by them, and is the activity removed with other impurities? "

About 4 years ago, in anticipation of the time when the disposal of liquid radio-active wastes became a problem, the Ministry of Health Water Department (which has now been transferred to the Ministry of Housing and Local Government) sponsored research on the behaviour of radio-active contaminants during sewage treatment. More recently, the Water Pollution Research Laboratory have installed facilities for similar work on water treatment. There is plainly a wide field for investigation which is likely to give some much-needed information, not only on what happens to radio-activity during treatment, but also on the treatment processes themselves. In other fields, radio-active tracer elements, as they are called, have proved a valuable research tool.

The answer to the question whether treatment processes will be affected by radio-activity is quite reassuring. Bacteria and lower organisms generally are far more resistant to radiation than we are, and work generally indicates that the concentration of radio-activity needs to be several powers of 10 greater than life-time tolerances before they are affected at all. The possibility that such conditions will ever prevail in this country is very remote indeed and need not be considered further now.

On the question of the effect of purification processes on radio-active constituents no general conclusion can be stated; there are so many possibilities. The active constituents may be phosphorus, iodine, or gold from hospitals, the numerous fission products from atomic piles, and miscellaneous isotopes from research and industrial establishments. The behaviour of all these will not be the same, and will be influenced by the type of liquid in which they are carried, whether it is a purely domestic sewage or one highly charged with trade wastes, or whether it is a soft moorland water or a hard river water. Nevertheless, the following general considerations assist in predicting what will happen. The first is that active and inactive isotopes of an element are chemically identical, and if present in the same chemical form they will behave in the same way during chemical or biological treatment. Thus, any radio-active calcium salt will behave in an identical manner to the inactive salt and will not be removable by any treatment short of complete softening. The second consideration is that in the extremely minute concentrations in which radio-active substances are normally present, adsorption on chemical or biological flocs becomes an important factor, particularly of elements of higher valency. By employing these means, contamination with elements which are normally quite absent from water-supplies may prove readily removable.

The work we have referred to on sewage treatment was published in a Paper by E. H. Belcher,[1] read before the Institute of Sewage Purification in June, 1951. It dealt with active iodine and active phosphorus, which are very widely used for therapeutic purposes, and with others selected in the hope of including elements with a wide variety of behaviour. Very little active material of any kind settled out during primary sedimentation, the sludge having little adsorptive capacity. During biological treatment, however, both by percolating filters and by activated sludge, a high percentage of cobalt suffered adsorption from the effluent and it is known that the same is true of yttrium. A good deal of phosphorus enters the secondary sludge, though some of it is released to the effluent at a later stage. Radio-sodium, radio-bromine, and radio-iodine were generally not greatly adsorbed, but passed forward with the effluent. These findings are in accord with expectations on the considerations mentioned above.

The work on water-treatment processes in this country is being carried out by the Water Pollution Research Laboratory and it would be unfair to them to anticipate publication. It may be said, however, that flocculation with alum or ferric salts, as expected, does not remove the monovalent elements iodine, caesium, and presumably others to a significant extent. On the other hand, phosphorus and yttrium are easily removed. An interesting finding with slow sand filters is that certain elements may be held back in the biological film and released at a later date. This is not in itself purification, but it will have the same effect if the isotope is short-lived, for it allows decay to take place. It may prove possible in the case of some elements to modify existing water-treatment processes so as to deal with them effectively. For instance, a suitable precipitant may be added to the water prior to flocculation and the subsequent floc will then enmesh the particles of insoluble active salt and remove them from the water. Such processes may have to be adopted in certain circumstances, but their scope is limited by the facts that normal purification must not be disturbed in any way, that the reagents must themselves be non-toxic at the concentrations used, and that the quantities of water requiring treatment are so vast. It would be quite impracticable, for instance, to adopt a process depending for its efficient operation on a *pH* of 11.

These limitations are much less severe when smaller volumes are in question as, for instance, with effluents before discharge from an establishment. Here the *pH* can be varied to suit the process, provided it is adjusted again later, and the choice of reagents is wider. The problem is simply one in chemistry and it is not difficult to devise means for removing activity due to most elements. Besides precipitants, the use of ion-exchange methods may also be feasible and in some cases distillation may be resorted to.

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[1] "Experimental Studies on the Fate of Radio-active Materials in Sewage Treatment" J. Inst. Sew. Purn, Part 3, 1951.

One point which must not be lost sight of is that no method of purification actually destroys radio-activity. There is no known method of doing so except by allowing it to decay at its natural rate. All that purification does is to concentrate the activity in a small volume of liquid or sludge, leaving the treated water relatively free from it.

This brings into prominence the question of the safety of sludges and the related question of the build-up of active slimes and deposits in sewers. There is a certain amount of information on these points, but much more can often be gained by calculation. May we illustrate this in relation to phosphorus and sewage sludge ?

No one eats or drinks sewage sludge, and active phosphorus does not give out gamma rays, so that there is no danger in simply being near the sludge. What can happen is that the sludge can be used as a fertilizer and the crop grown on it eaten by man. Would this constitute a danger ? Let us assume a sewage loaded up with active phosphorus to the life-time tolerance figure for drinking water. The reason for that assumption will be apparent later. The sewage will, of course, contain inactive phosphorus, as all sewages do, and a concentration of it of 10 parts per million is reasonable. Part of both types of phosphorus will be retained in the sludge ; on the principle of pessimistic assumptions let us suppose that all of it goes with the sludge. The sludge is then used as a fertilizer, and let us assume that the crop gets all its phosphorus from the sludge. Now there will be a time interval between the arrival of the phosphorus at the sewage works and the eating of the crop, and this can hardly be less than 6 months. In many cases it is, of course, more than 6 months before the sludge leaves the works, though sometimes it is less. During that 6 months the activity will decay by half every 14 days, for that is the half-life of active phosphorus. Now assume that a man eats nothing else but food grown on this sludge for a year. This is carrying pessimism to absurdity, of course, but let that pass. Now calculate how much active phosphorus he would ingest. The answer is only one-fortieth of the quantity he would take in drinking water loaded to the life-time tolerance level for beta emitters other than strontium and calcium. We may, I think, conclude that there is no danger in the sludge. Many possibilities can be covered by calculations such as this.

We must now return to the problem of the control of active effluents. It is solved by the application of two principles. The first is that, apart from natural radio-activity, which has always been with us, there can be no more activity in a river than is discharged into it. This enables us to turn our attention from the river itself to the effluents discharged into it, which, being much smaller in volume, will be more concentrated. The increased activity is easier to determine and at the same time makes a certain amount of analysis of active elements not impracticable. The second principle is that to which we have given the name of "pessimistic assumption," which in this respect means that all activity which cannot be proved not to be dangerous must be assumed to be dangerous.

Let us illustrate by reference to the discharge from the Atomic Energy Research Establishment at Harwell. There the problem was in its most acute form, because, A.E.R.E. being a research establishment, the effluent can be a very complicated mixture and the River Thames, to which the effluent is discharged, must be maintained safe for drinking by millions of people. Maximum concentrations of different kinds of activity have already been given and it has been pointed out that these are mutually exclusive; that is, if one class is present in a given proportion of its maximum the others must be correspondingly reduced. A formula was derived to take this into account quantitatively. A mean value for the flow of the river at the discharge point was ascertained and an allowance made to cover possible active discharges previously made into it. From these an overall monthly figure was calculated and Harwell undertook not to exceed it. Each batch of effluent is analysed for activity. If strontium and calcium are not separately determined, then all the beta activity is assumed to be due to these elements. If these elements are separately estimated by a method we know cannot underestimate them, then these elements are assessed separately and the remaining beta activity assessed against the higher tolerance level of concentration—that for other beta emitters. Duplicate samples of all discharges are retained for checking by this Ministry's radiochemical inspector who also has authority to take samples for himself at any time and without notice.

We have not given all the details in this brief account, but it can be said that discharges have been made over a period of nearly 4 years and that it has been shown that they can be adequately controlled and supervised.

We must now turn to the very many other effluents containing activity which are not so controlled and supervised: iodine 131, phosphorus 32, and smaller quantities of other elements from hospitals. Naturally these eventually find their way into sewers. Radio-active elements are also used by various establishments for research purposes and some of these, at least, will be discarded into the drains after use. Yet no sewage authority samples its sewage regularly for activity determinations; indeed, the provision of apparatus and staff at sewage works for such a purpose can hardly be contemplated with present financial and manpower shortages. Do we know that these are safe; and if so, how?

Once again we apply the two principles, with the first one modified. Instead of saying that a river cannot contain more activity than is put into it, we say that a sewage cannot contain more activity than that which enters the district which produces it. Radio-active isotopes are not made in many places; the suppliers are Harwell and its associated establishments. A record is kept of all shipments and their destinations and every month we receive a copy of the record. We are then in a position to know how much activity, and what sort, is used in every sewerage area and to calculate the relationship between the activity and the volume of sewage. So long as we know that, on the average, the sewage of a district *cannot* be contaminated up to life-time drinking-water tolerance level we feel that there is no threat to safety and that no further control is justifiable. There are, you will note, two considerable safety factors in this attitude. The first one is that much radio-active decay might take place both before disposal and later in the river, and the second is that sewage, even treated sewage, is not drunk (certainly not for a life-time) without further dilution and further treatment. It is, however, the fact that, apart from one centre of population, the amount of activity now being supplied is not sufficient to contaminate any area's sewage to drinking-water standard. In that exceptional case much of the activity is very short-lived and the stream to which the sewage is discharged is not used at all as a source of drinking water.

How long this situation will be maintained we cannot tell. The use of radio-activity is on the increase and dangers at present non-existent may become real and have to be dealt with. In some, perhaps many, cases it would be unreasonable to insist on drinking-water standard in sewage. For the London sewage, for instance, some other less stringent standard would be sufficient. But whatever the standard there must in every case be a proper limit beyond which waste radio-activity should not be disposed of to sewers, or to a river; and the same applies to other methods of disposal we have not considered to-night. Control will be necessary.

It is at this point that we must mention the Radio-active Substances Act, 1948. By virtue of Section 5 of the Act, a Minister may make regulations for the purpose, amongst other things, of securing a safe disposal of waste radio-active substances, and a breach of the Regulations is an offence.

The Act leaves the appropriate Minister undefined and it will require an Order in Council before we know who he is. Presumably no regulations will be made until after that. A good deal of thought is being given to the type of regulation which would be most suitable, which would at the same time safeguard all interests and yet not hinder the development of proper and profitable use of radio-activity. We hope, however, that we have made it plain that at the present time, so far as sewage and water are concerned, there is no urgent necessity for regulations. How long this state of affairs will last we shall not attempt to forecast.

CORRESPONDENCE

On a Paper published in
Proceedings Part III, April 1952

Public Health Paper No. 4

" Some Problems in the Disposal of Industrial Effluents and Domestic Wastes "†

by

John Thornton Calvert, M.A., M.I.C.E.,

and

Peter Maurice Amcotts, A.M.I.C.E.

Correspondence

Mr A. W. Shilston observed that open sludge-drying beds had been installed in the sewage-disposal scheme at Crawley New Town. Such a method of drying sludge was undoubtedly inefficient and Mr Shilston wondered if consideration had been given to alternative methods of sludge drying.

Whilst the majority of engineers who had had dealings with sewage disposal matters agreed that the use of open beds was obsolescent, many held the view that it was still the most economical method available and more modern methods were not justified.

At Crawley it might be inferred from the Authors' remarks that, owing to the need for maintaining high standards of purity in the watercourses, it was necessary for sludge beds to be of impermeable construction, and not to permit foul liquors to penetrate into the subsoil. It would follow that they were fairly expensive to construct. In addition, the Authors had stated that sludge-drying beds had been provided on the basis of 1 square yard of bed for every six persons. Mr Shilston suggested that on that basis the beds would be of insufficient capacity and that a figure of 1 square yard per three persons would have been more realistic. Taking those factors into account it might be that, in comparing the cost of constructing and operating sludge beds on the revised basis, with corresponding figures for more modern methods of drying, the differences would not have been great. A rather obvious, but nevertheless significant disadvantage of adequate-capacity sludge beds was the necessity of sterilizing a large area of land in accommodating the beds.

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† Proc. Instn Civ. Engrs, Part III, vol. 1, p. 141 (Apr. 1952).

Although not specifically mentioned in the Paper, Mr Shilston assumed from the drawings that the digesters contained internal heating coils for raising the temperature of the sludge. Did the Authors consider such a system preferable to the use of external heat exchangers ?

Whilst realizing that constructional details were somewhat outside the scope of the Paper, Mr Shilston asked what special precautions had been taken in the construction of circular concrete tanks to lessen the effects of shrinkage and what approximate length of panels had been employed during the placing of the concrete walls.

Mr R. F. B. Gaudin referred to the Authors' statement regarding the importance of research in " the problem of the return of sewage or sewage sludge to the land as a fertilizer, which assumes greater importance with the dwindling food resources of the world."

At the end of the 19th century, domestic sanitary arrangements frequently consisted of a shed at the end of the garden containing a seat with a pit beneath. Such an arrangement was not confined to working-class dwellings, nor was it very exceptional. The pit was emptied from time to time and the contents found their way to the garden as an ingredient of compost ; frequently no other form of fertilizer was used. The people lived well on healthy fruit and vegetables.

During the evolution from that kind of thing to the present, the ruling policy, with a few exceptions, had been to make the effluent harmless and safe to turn into the stream or river. It appeared to have been treated as a disposal of a waste without much regard as to how that waste could be changed to wealth.

At present, more and more interest was being taken in methods of using those wastes for the ultimate benefit of agriculture. Schemes to that end existed in Dumfries-shire, Edinburgh, and elsewhere but even those did not go far enough, for it was probable that many minerals most valuable to plant growth evaded capture and found their way in solution to the river or sea. The soil was thus progressively denuded of most valuable minerals in just the form required for plant life. It was a leaky tank system and although a leaky tank was better than none, it was not yet good enough.

Bearing that in mind it would doubtless be of interest to put on record a note on the Jersey proposals in so far as they dealt with saving the minerals which would be lost. There the effluent would be turned into a lagoon planted with Water Hyacinth which would absorb those minerals. The plants would be cut from time to time and used with seaweed, sludge, and treated domestic wastes to form a fertilizer containing all those ingredients, including trace elements, in a form acceptable to soil life, and through it would produce vigorous and wholesome plants.

Mr Gaudin believed, therefore, that there would eventually be a complete return of all organic and mineral wastes to the land and so revert to the principle of the " cycle of life " —the principle of returning all organic wastes with their associated minerals to feed and foster healthy plant organisms and through them healthy animals and men.

The Authors, in reply, disagreed that open sludge-drying beds were inefficient except in the use of land to which Mr Shilston had referred. Consideration had been given to other methods of sludge dewatering but in the Authors' view it was uneconomical to employ expensive chemicals and power to remove water when nature would achieve the same result by the natural process of evaporation. At very large works the land or labour required might be beyond the local resources, necessitating the use of mechanical dewatering, and also when dealing with large populations the cost of mechanical processes compared more favourably with drying beds.

The Authors preferred external heat exchangers to internal heating coils, and at Crawley the former method was being adopted. Two digestion tanks and a consolidation tank were to be constructed in the extensions to the works to be undertaken in the near future and the sludge heaters and circulating pumps would be housed in an extension to the power-house building. The drawings incorporated in the Paper were preliminary lay-out drawings, not intended to show details.

The Authors agreed that the effects of shrinkage in concrete tanks were sometimes troublesome, but in the case of the circular settlement tanks, which were 60 feet in diameter, no special precautions had been taken in placing the concrete walls. Half the circumference had been placed in one operation and no contraction cracks had been observed.

The Authors were interested to read Mr Gaudin's account of the Jersey proposals and agreed that the recovery of the fertilizer value of sewage was very desirable so long as it could be attained in a reasonable space and without nuisance. They would, however, deprecate the return to the old sewage farm in spite of the fact that it had achieved that object.

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